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THE UNIVERSITY OF ALBERTA
A STUDY OF PARTICLE RE-ORIENTATION IN THE
DIRECT SHEAR TEST

by



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A THESIS
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The undersigned certify that they have read, and
recommend to the Faculty of Graduate Studies for acceptance,
a thesis entitled A STUDY OF PARTICLE RE-ORIENTATION IN THE
DIRECT SHEAR TEST submitted by David Samuel Matheson in
partial fulfilment of the requirements for the degree of
Master of Science.

ABSTRACT

An investigation of the behaviour of granular media during direct shear was performed by conducting direct shear tests upon artificially prepared calcite sands. The samples, at the end of shear, were impregnated with Carbowax 6000, sliced, mounted on a glass microscope slide, and ground to produce translucent sections about 0.01 inch thick.

The orientation of the constituent calcite particles in each slide was studied. It was found that elongated particles rotate during shear to form a preferred orientation in the direction of shear. In samples of medium density, no shear zone as such was apparent; the orientation of the particles extended over most of the depth of the sample. Experimental results indicated that increased density inhibited particle rotation while increased normal load enhanced the degree of particle orientation visible at a given horizontal displacement. The experiment results were found to confirm, qualitatively, theory for the motion of particles in a sheared viscous fluid. It is suggested that granular soils behave, during shear, not unlike a viscous fluid.

Experimental results showed that at low normal loads the direct shear sample failed along two arc-shaped failure surfaces in the upper-trailing and lower-leading portions of the sample. A theoretical study of the characteristics of

ideal spheres in direct shear indicates that these failures mark the boundaries of zones in which no intergranular forces, due to the shear force or its reaction, can act. Further experimental results show that, at low normal loads, only a portion of the surface of the sample dilates during shear.

It is postulated that peak strengths as measured in the direct shear test are apparent strengths and are not a true measure of the shearing resistance of the sample along a planar failure surface.

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CHAPTER I

INTRODUCTION

1.1 GENERAL OUTLINE OF THE SHEAR STRENGTH OF SOILS

The shear strength of a soil is perhaps, the most important property which the engineer wishes to know in many problems. It is unfortunate that knowledge of how and why soils deform under load and fail is somewhat limited. Indeed, there exists no clear and satisfactory definition of what is meant by "failure" of a soil sample.

The most commonly used criterion in current engineering practice is the Mohr-Coulomb failure criterion. The shear strength of a soil (τ_f), using effective strength parameters, is given by:

$$\tau_f' = c' + \sigma' \tan \phi'$$

where c' and ϕ' are the effective cohesion and angle of shearing resistance (or internal friction) respectively, and σ' is the normal pressure on the failure plane.

The validity of the Mohr-Coulomb failure criteria has never been proven. While an excellent first approximation of the shear strength of soils, the Mohr-Coulomb failure criteria does not accurately predict all the facets of a soil's behaviour during shear. The Mohr-Coulomb failure

criteria is essentially an empirical relationship; there is no theoretical proof or derivation of it. It has never been conclusively proved experimentally. Thus, if this criterion is not valid, a good deal of effort and time in current research is misdirected.

A number of anomalies exist between the assumptions of the Mohr-Coulomb criterion and the actual behaviour of soil during shear. Soils dilate during shear, form failure planes at unpredicted orientations and yield different strengths as the intermediate principal stress is varied. None of this behaviour is predicted by the Mohr-Coulomb criterion.

It has been shown (Seed and Chan, 1959) that the orientation of clay particles has a marked effect upon the properties of clays. Skempton (1964, 1965) has shown that shear zones, or zones of oriented particles, occur after a large horizontal displacement in a direct shear test or in the shear (failure) zone of an earth movement.

The only laboratory test one can use to measure the strength associated with an orientated shear zone is the direct shear test. The disadvantages of this test are well known (Terzaghi and Peck, 1948). The direct shear test, however, is becoming increasingly widely used as the residual angle of shearing resistance becomes more widely used in stability analyses.

1.2 PURPOSE OF THIS RESEARCH

This research was primarily carried out to study the behaviour of the particles of a granular soil during the process of shear. The change in particle orientation of calcite grains in the direct shear test was studied by means of thin sections using a modification of Mitchell's (1955) technique. As the resulting sections were somewhat thicker than those Mitchell used (30 microns) the term "section" will be used in this thesis rather than the term "thin section".

Secondary objectives of this research were a study of the direct shear test itself and the development of a technique to make sections of granular media.

1.3 SCOPE OF THE THESIS

It was originally intended to study the process of shear failure in cylindrical silt samples by means of unconfined compression tests. This proved unsuccessful and details of the tests on silt, and of direct shear tests upon a natural clay shale, are given in APPENDIX A.

Samples of artificially produced calcite sand were sheared to differing horizontal displacements in a direct shear box. The samples were carbowaxed, extruded from the shear box and sectioned. Full details are given in Chapter IV. The resulting sections were photographed and analyzed. The effects of varying the normal load, sample density and grain size distribution were studied. Details in experimental

technique were varied and are reported upon.

The direct shear test was studied from both test results and the theoretical behaviour of ideal granular media. Experimental results showed that elongated particles rotate during shear and form a preferred orientation parallel to the direction of shear. The experimental results are shown to confirm the theory derived by Gay (1966, 1968a, 1968b) for deformable particles in a sheared viscous fluid.

It is postulated that the peak shear strength as measured in the direct shear test is an apparent strength which is somewhat higher than the "true" peak strength that exists for a given soil sample. Qualitative data to support this hypothesis are presented.

CHAPTER II

LITERATURE REVIEW

2.1 SCOPE

A study of the literature published on the shear strength of soils shows that a good deal of work remains to be done in this field of soil mechanics. How and why a soil fails under an imposed stress system is still an open question. What constitutes failure is undefined or ambiguous.

Studies carried out on ideal systems and simplified models will yield a limited amount of information as the results are only strictly applicable to these very simplified systems. A method must be used where the movement of actual soil particles during shear can be studied. Thin sections appear to meet this requirement to some degree.

2.2 EXISTING STRENGTH THEORY

In 1776 the French military engineer, C. A. Coulomb, formulated an empirical relationship for the shear strength of soils. This relationship is referred to as Coulomb's law and states that:

$$s = c + \sigma \tan \phi$$

where s is the shear stress in the plane of failure at the time of failure, c is the cohesion, ϕ is the angle of internal

friction and σ represents the stress normal to the failure plane.

In the late nineteenth century Otto Mohr, a German engineer, generalized Coulomb's law. Mohr, as reported by Leonards (1962), postulated that a material will fail when the shearing stress on the plane of failure is a unique function of the normal stress acting on that plane. Mohr's failure theory is illustrated in FIGURE II.1. Thus the equation $\tau_f = c + \sigma \tan \phi$ is the equation of a rupture line and is referred to as the Mohr-Coulomb failure criteria. When the state of stress upon the potential failure plane is below this rupture line (Circle 1) the soil is stable. When the circle of stress reaches the rupture line (Circle 2) failure of the soil occurs. No point representing a combination of normal and shearing stress can fall above the line of rupture.

The Mohr-Coulomb failure criteria is the most widely used current strength theory. It has been modified by Terzaghi to the form $\tau'_f = c' + \sigma' \tan \phi'$ where effective strength parameters are used and $\sigma' = \sigma - u$ where u represents the pore pressure. The original form of the failure criteria is used in cases where no water is present in the soil.

It is interesting to consider the basic assumptions of the Mohr-Coulomb failure criteria. Mohr assumed that the soil can fail only by slippage along discrete failure planes. No volume change is assumed to occur and the line of rupture is assumed independent of the means by which it is obtained

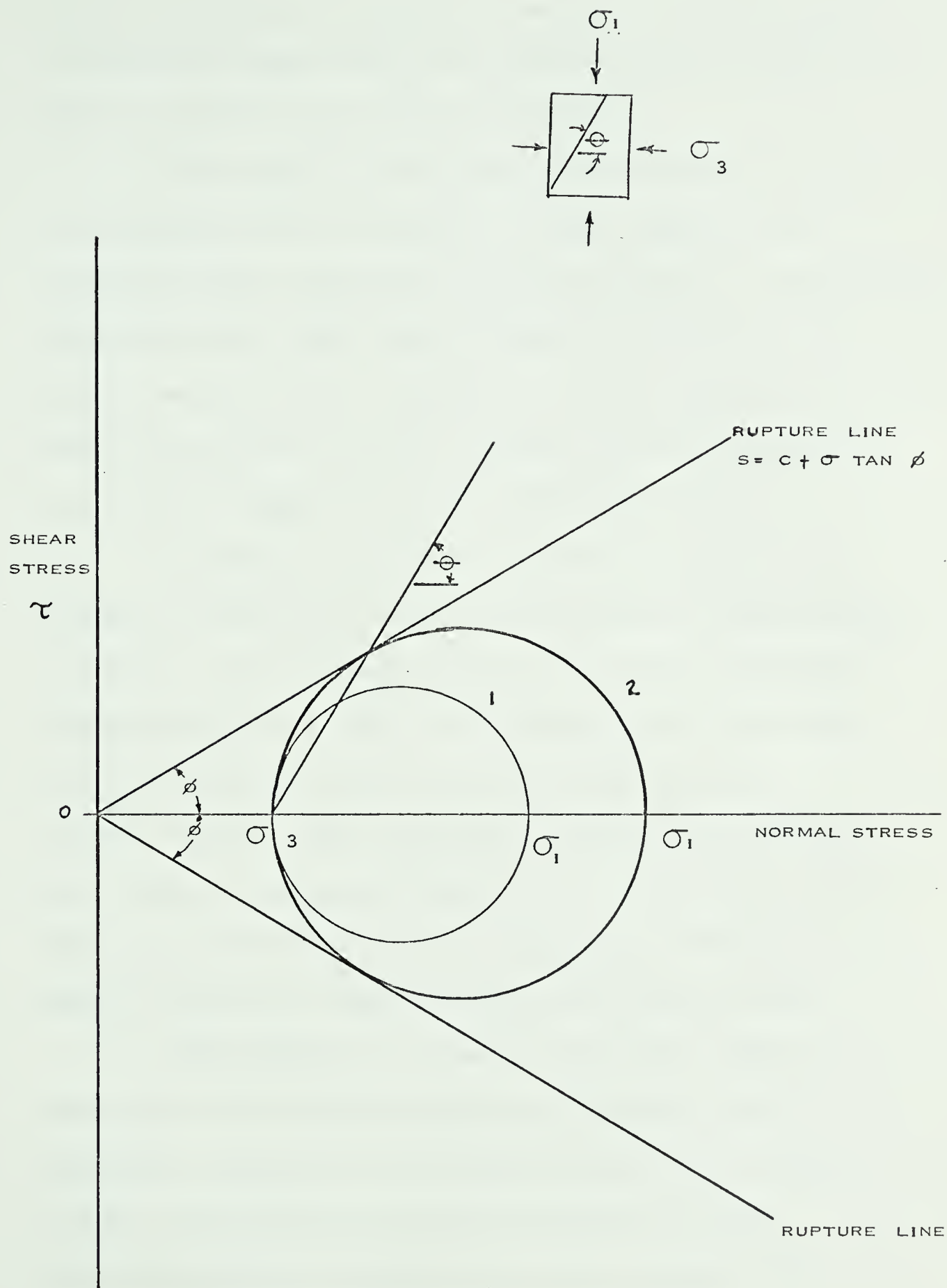


FIGURE II . I - THE MOHR - COULOMB FAILURE CRITERIA
[AFTER LEONARDS , 1962]

and the intermediate principal stress (σ_2). The orientation of the failure planes should be predictable from the plot of failure conditions as is shown in FIGURE II.1.

The validity of the Mohr-Coulomb failure criteria has been questioned when applied to clays; however, it is generally thought applicable to granular materials. Kirkpatrick (1957) and Geuze (1962) have concluded that the failure of granular materials can be predicted with good results by the Mohr-Coulomb failure criteria. Evidence of other researchers, as is shown in TABLE II.1 yields conflicting results.

A drawback in studying results of strength testing of soils is that there is no clear definition of what is meant by failure. It is commonly taken as the peak of the stress-displacement curve (shown in FIGURE III.1). The peak of the principal effective stress ratio plot is often used in triaxial testing. From a mechanistic point of view, failure may be taken as irreversible deformation of the system. These terms are not synonymous and thus a good deal of confusion is added to the study of shear strength and failure in soils.

From TABLE II.1, it can be seen that a number of facts about the failure of soils are at variance with the fundamental assumptions of the Mohr-Coulomb failure criteria. In fact, practically all the basic assumptions of the Mohr-Coulomb failure criteria are in poor agreement with the observed behaviour of soils.

The volume expansion (dilation) of the sample

(as shown in FIGURE V.1) occurs in the early stages of shear and adds to the shearing resistance of the sample. Taylor (1948), Bishop (1954) and Newland and Allely (1957) have attempted to prove that sample dilation is the cause of the peak of the stress-displacement curve. None of these approaches were entirely successful although Newland and Allely (1957) found good agreement between theoretical and experimental results for triaxial tests upon lead shot for cell pressure over 20 p.s.i.. The observations that sample expansion begins before the peak stress is reached and the volume increases linearly over most of the displacement range is at variance with the theory developed by Newland and Allely. None of these three approaches can be said to satisfactorily explain why a peak exists in the stress-displacement curve for a dense granular material.

The concept of sand being only able to slide along rupture lines is fundamental to the Mohr-Coulomb failure criteria. As pointed out by Jennings and Kirchmann (1962), Mohr's hypothesis of failure depends upon the fact that failure takes place upon slip planes when the stresses exceed a certain maximum value. Rowe (1962) points out, for triaxial tests upon perfect spheres, that slip planes are not the cause of failure but occur past the peak in the stress-displacement curve. The slip planes are the later result of failure. The implications of this most important point will be discussed in Chapter VI. Cornforth (1964) shows that failure planes are thin and well defined in dense samples of sand. In looser

samples a thicker "shear zone" results which is poorly defined.

The effect of the intermediate principal stress (σ_2) upon the shear strength of cohesionless soils is somewhat ambiguous. Saada (1963) gives a good resume of the conflicting experimental results and shows that a considerable variation in the angle of shearing resistance results from a change in σ_2 .

Considerable difference in opinion exists as to the effect of the kind of test upon the value of the angle of internal friction, Habib (1953), Peltier (1957) and Haythornwaite (1960) found that triaxial compression tests yield larger values of the angle of shearing resistance than triaxial extension tests. Henkel (1959) and Ko and Scott (1968) found the reverse; extension tests yielded higher results. Cornforth (1964) found similar results from triaxial compression and extension tests. Bishop (1966) states that Mohr-Coulomb underestimates the angle of shearing resistance by up to 4° for plane strain tests upon dense sand. Nash (1953) found that results from direct shear and triaxial tests upon fine sand differ and that the density of the sample plays a major role in the variation.

The Mohr-Coulomb criteria predicts the orientation of failure planes as shown in FIGURE II.1. Most authors (Gibson, 1953; Hvorslev, 1960; Saada, 1963) point out that the measured orientation of failure planes in the triaxial test is in poor agreement with predicted values. This is usually attributed to "stress concentrations".

TABLE II.1

ASSUMPTIONS AND VALIDITY OF MOHR-COULOMB FAILURE CRITERIA FOR SOILS

ASSUMPTION	SOURCE OF ASSUMPTION	EXPERIMENTAL EVIDENCE	SOURCE OF EVIDENCE
1. No volume change during shear	Rowe (1962)	Invalid Invalid	Reynolds (1885) Rowe (1962)
2. Sand can only slide along rupture lines	Hansen (1961) Jennings and Kirchmann (1962) Leonards (1962)	Untrue at peak	Rowe (1962)
3. Line of rupture independent of means obtained	Rutledge (1940) Jennings and Kirchmann (1962) Leonards (1962)	Valid Invalid Valid Invalid Invalid Invalid	Kirkpatrick (1957) Ko and Scott (1968) Bishop and Eldin (1953) Habib (1952) Haythornwaite (1960) Henkel (1959)
4. Line of rupture independent of intermediate principal stress	Rutledge (1940) Jennings and Kirchmann (1962)	Invalid Invalid Invalid Invalid Valid Invalid Invalid Valid	Habib (1953) Rutledge (1940) Hvorslev (1960) Kirkpatrick (1957) Kjellman (1936) Wu, Loh & Malvern (1963) Cornforth (1964)
5. The orientation of the rupture planes can be predicted	Rutledge (1940) Jennings and Kirchmann (1962) Leonards (1962)	Doubtful Doubtful Invalid Poor agreement Invalid	Rutledge (1940) Hvorslev (1960) Saada (1963) Jennings and Kirchmann (1962) Gibson (1953)

Considering the preceding experimental observations, it is apparent that the Mohr-Coulomb failure criteria does not perfectly describe the failure of soils. It is merely a good first approximation of the behaviour of soil during shear. Unfortunately, no other practical alternate criteria yet exists. Bishop (1966) points out that the use of the extended Mises or Tresca criterion would lead to a very substantial overestimate of strength for some conditions. Bishop (1966) states that:

"The experimental results, however, strongly support the Mohr-Coulomb criterion, and we must, I feel, accept the Mohr-Coulomb as being the only simple criterion of reasonable generality."

There can be no doubt about the validity of the later assertion by Bishop about the Mohr-Coulomb failure criteria being the only current practical strength criteria. The former assertion, regarding experimental results strongly supporting Mohr-Coulomb, is somewhat doubtful in view of the points covered in TABLE II.1.

2.3 APPLICATION OF STRENGTH THEORY

It can be argued that a great number of foundations and embankments have been successfully designed using the Mohr-Coulomb failure criteria as a basis. However, it must be pointed out that large factors of safety are commonly used in current design practice. For embankments 1.50 is accepted as a working design factor of safety (Bishop and Bjerrum, 1960). In foundation designs a factor of safety of 3.0 is

commonly used (Terzaghi and Peck, 1948). The use of large factors of safety is necessary when dealing with a material which can have widely varying properties from samples taken within a few feet of each other. However, it is possible that the large factors of safety presently in use could be reduced if the failure mechanism of soils were better understood. If the Mohr-Coulomb failure criteria is not correct, use of the current large factors of safety presently in use could be reduced if the failure mechanism of soils were better understood. If the Mohr-Coulomb failure criteria is not correct, use of the current large factors of safety would tend to obscure the fact as relatively few failures would occur.

A primary use of the concept of shearing strength is in the field of stability analysis. Use is made of the Mohr-Coulomb criteria:

$$\tau'_{.f} = c' + \sigma' \tan \phi'$$

to calculate the strength of the soil along a potential failure surface. From a comparison of this strength with the imposed stresses due to forces tending to actuate the slide, a factor of safety is obtained. However, a study of many failures indicates that one cannot assess the stability of an intact earth slope with any degree of reliability (Peck, 1967).

The strength parameters arrived at in the laboratory, c' and ϕ' , have been shown to be a function of test conditions (Hvorslev, 1960). Therefore considerable

work has been done to adjust these parameters to yield factors of safety of unity in cases where slopes have failed. Examples are Hardy, Brooker and Curtis (1962) and Skempton (1964).

Skempton (1964) proposed that for long term stability, the cohesion of a clay tends to zero and the residual strength of the stress-deformation curve be used to calculate a residual angle of internal friction. This approach has been shown to apply in some cases but has not been successful in others (Skempton, 1964, Hayley, 1968).

Thus it would appear that variation of techniques of testing and analysis, in hope of finding some universal means which will give a factor of safety of unity, is unrewarding. A detailed study of how and why failure occurs in soil could lead to a better understanding of stability problems and, if necessary, to a revision in strength theory and testing. Use of lower factors of safety and more economical designs could result from a better understanding of failure in soils.

2.4 USE OF THIN SECTIONS

Lambe (1960) stated that:

"The development of a technique which will permit the visual observation of adjacent particles in any soil without disturbing the particles will be a powerful stimulant to mechanistic research."

Thin sections have been used since the turn of the century by geologists to study the mineralogical

composition of rocks. Basically, the sample face is ground with both coarse and fine grit and then glued to a glass microscope slide. The rock is then ground away on a lapping wheel until the sample is thirty to forty microns thick. The sample is translucent at this thickness and readily passes light. It is studied under a polarizing, or petrographic microscope and by the use of petrographic techniques, constituent minerals in the sample can be identified. Details of procedures are given by Bloss (1966).

For many years engineers have been interested in the structure of soils, especially of clays, as a means of explaining their behaviour. Soils are composed of mineral grains and voids. To enable sections of a soil to be studied optically some binder must hold the soil particles in the original position otherwise the slicing and preparation of the sample will disturb the mineral grains.

Rosenqvist (1955) impregnated clay with sulphonated alcohol and was able to cut sections down to two microns thickness with a biological microtome. This method, however, would not be suitable for use in silts or sands as the microtome blade would not be able to cut quartz particles.

Mitchell (1956) introduced the use of Carbowax 6000 to fill the voids in a soil. This polyester glycol (produced by Union Carbide Company) is hard at ordinary temperatures, melts at about 55°C. and is entirely water soluble. Since then it has been used in the study of clay samples by Quigley and Thomson (1966) and Morgenstern and Tchalenko (1967).

Both of the above papers agree that the shrinkage of the carbowax is slight and does not appreciably affect the fabric of the clays.

Morgenstern and Tchalenko (1967) carried out direct shear tests to varying horizontal displacements upon samples of kaolin. The authors concluded from their study that the center of the sample is in simple shear and that the process of shear is a matter of considerable complexity.

2.5 THE DIRECT SHEAR TEST

The direct shear test and the triaxial compression test are the two main means of determining the shear strength parameters of a soil. The direct shear test was in common use before the triaxial test. It has, however, a number of disadvantages which led to the preferential use of the triaxial test. These are:

1. Progressive failure from the ends of the sample (Hvorslev, 1960).
2. Uneven distribution of shearing stresses throughout the sample (Roscoe, 1953).
3. Measurement of sample volume changes is more difficult (Lambe, 1951).
4. Difficulty in controlling flow of water to and from the sample and measuring pore pressures (Lambe, 1951).

For these reasons triaxial testing of soils predominated until Skempton (1964) introduced the use of the residual angle of internal friction (ϕ'_r). This parameter is not obtainable from the triaxial test because of the large amount of sample deformation at large displacements. There has been therefore a renewed interest in direct shear testing.

CHAPTER III

THEORY

3.1 THE EFFECT OF PARTICLE ORIENTATION UPON STRENGTH

It is well known that the structure of a soil has a marked effect upon its properties. Mitchell (1956) and Seed and Chan (1959) have shown that the orientation of clay particles in natural and compacted clays has a marked effect upon the strength of these soils. In general, a clay with a random structure is stronger than one with a structure oriented toward the direction of shear.

Deformation imposed on a soil will produce preferred orientation in clays. This is strikingly shown by Morgenstern and Tchalenko (1967). Skempton (1964) shows that clays which have reached the residual strength, had developed a shear zone composed of oriented particles. Hence, it would appear that particle orientation could be the cause of "peak" and "residual" strengths. One would expect to find in the direct shear test a random structure at the peak being gradually replaced by an oriented structure after further horizontal displacement. This concept is illustrated in FIGURE III.1.

However, when a dense sample of cohesionless soil is sheared, dilation occurs. As the sample expands against

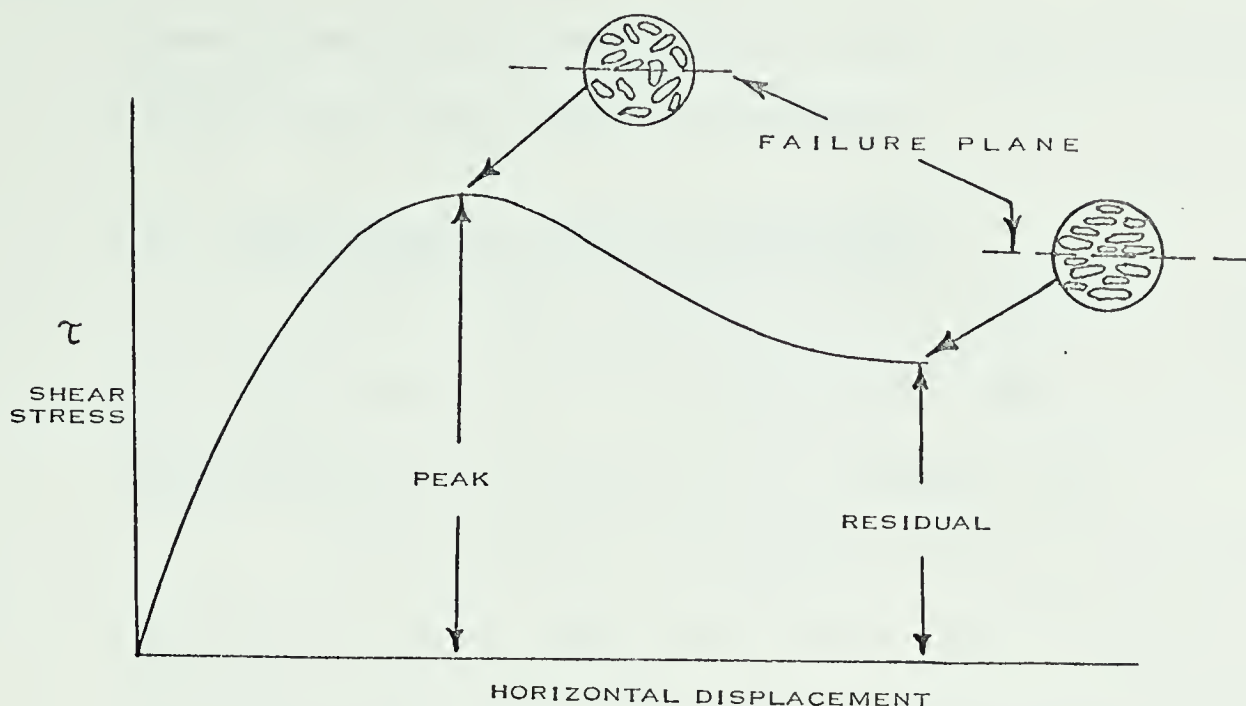


FIGURE III.1 - SKETCH OF PARTICLE ORIENTATIONS EXPECTED DURING DIRECT SHEAR

the normal load, work is done. Taylor (1948), Bishop (1954) and Newland and Allely (1957) have tried to explain the difference between peak and residual stresses on the basis of the work caused by this dilation. The work of Newland and Allely as well as the studies of "stress-dilatancy" carried out by Rowe (1962, 1963) and Horne (1964) assumes that particles move only by sliding. The possibility of particle rotation during shear has been ignored.

Particle rotation must occur during shear if an orientated structure is to form. Roscoe and Schofield (1963) report having observed particle rotations of as much as 15° in dense sand against the glass wall of a plane strain earth pressure apparatus. Arthur, as reported by Roscoe and Schofield (1963), postulated that anisotropy must develop in samples of

granular media subjected to shear tests. Particle rotation must occur if this anisotropy is to develop.

3.2 THE PROCESS OF PARTICLE ORIENTATION

Hill (1950) concluded that the center of a sample in a direct shear box is in simple shear. This concept has been confirmed by Morgenstern and Tchalenko (1967).

Work carried out by Skempton (1964, 1965) and Morgenstern and Tchalenko (1967) show that a failure zone forms in the direct shear test after a considerable amount of horizontal displacement. Along this slip zone the particles are in essentially perfect orientation. This concept is illustrated in FIGURE III.2.

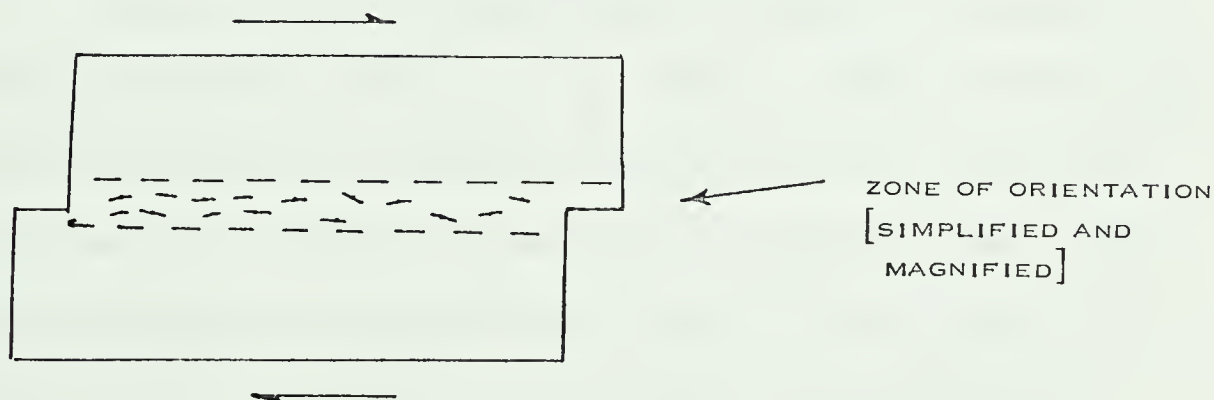


FIGURE III.2 - DIRECT SHEAR SAMPLE AT RESIDUAL STRENGTH

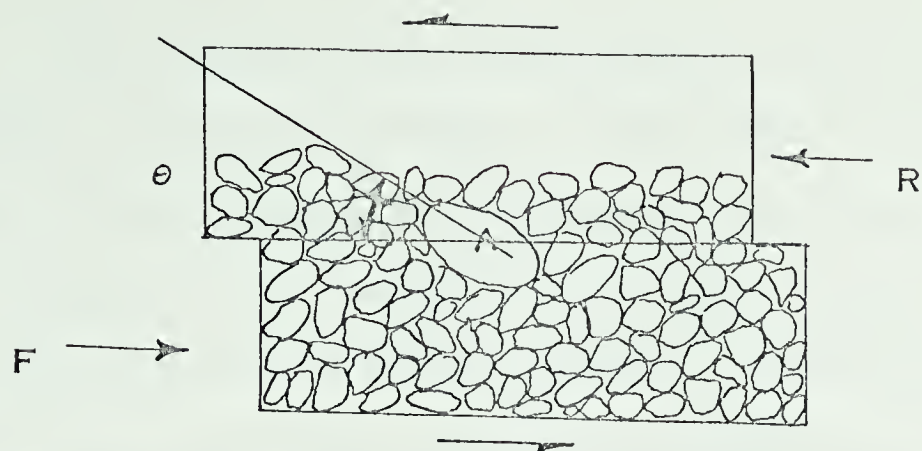
Particle orientation has also been noted by geologists. Bhattacharyya (1966) notes a number of relationships between the orientation of elongate mineral grains and the direction of fluid flow of magma and lava. In each case the long axis of the mineral grain was parallel to the direction of flow of the surrounding media.

Bhattacharyya (1966) noted a strong mineral lineation

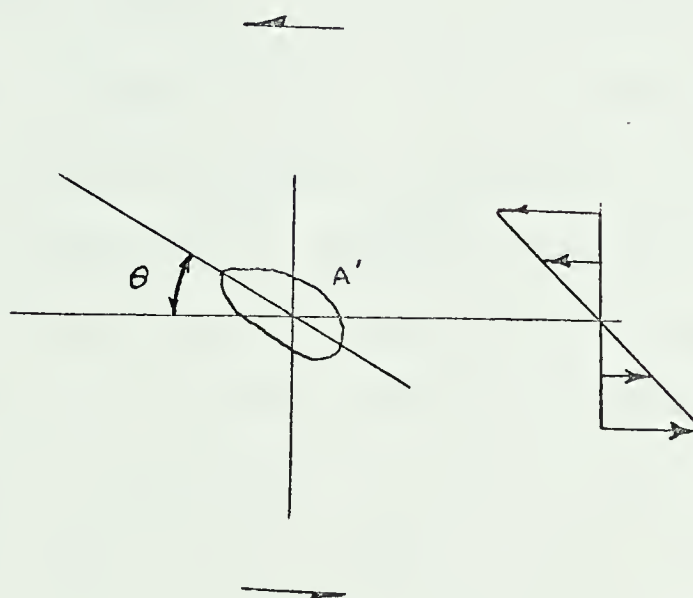
parallel to the direction of rock flowage when rocks were deformed in plastic flow. Cloos (1946) noted that prismatic grains were parallel to the direction of flow in a granite batholith. Crystals in a lava flow were also noted to align parallel with the direction of flow of the lava. Thus during shear, elongate mineral particles in a viscous fluid (lava or magma) or in a rock exhibiting plastic flow tend to become aligned with their long axis parallel to the direction of flow. It would appear that shear in rocks (and hence soils) and fluid flow of magmas yields analogous results; preferred orientation of elongate mineral grains in the direction of flow and shear.

The theoretical relationships of particle movement in a viscous fluid was established by Jeffrey (1922). Further work on particle motion in a sheared viscous fluid was carried out by Bhattacharyya (1966) for laminar shear and Gay (1966, 1968a, 1968b) for pure and simple shear. This work would appear to be applicable to the study of the rotation of particles of soil in the direct shear test.

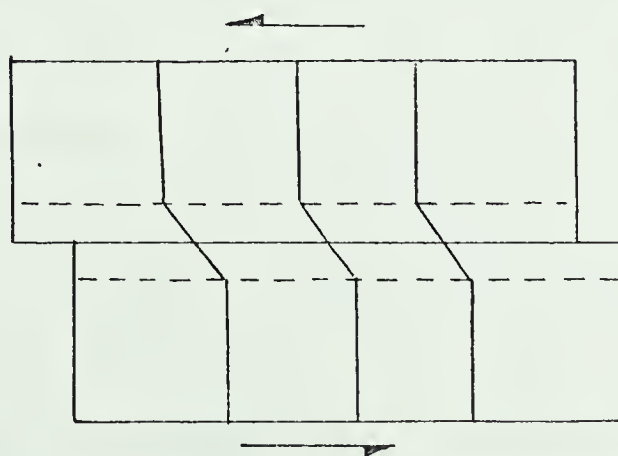
FIGURE III.3 (a) shows a sample of granular soil under simple shear. As can be seen from this diagram, particle A will tend to rotate in a counter-clockwise direction due to the forces imposed upon it by the particles in contact with it. This is an analogous situation to that shown in FIGURE III.3 (b) where a single particle, A', is immersed in a viscous fluid which is being sheared.



[A] A GRANULAR SAMPLE UNDER SIMPLE SHEAR IN THE DIRECT SHEAR TEST



[B] A MINERAL GRAIN UNDER SIMPLE SHEAR IN A VISCOUS FLUID



ZONE OF SIMPLE SHEAR
[AFTER MORGENSTERN
AND TCHALENKO , 1967]

[C] SIMPLE SHEAR ACTING UPON THE CENTER OF THE DIRECT SHEAR SAMPLE

FIGURE III . 3-SAMPLE BEHAVIOUR UNDER SIMPLE SHEAR

The motion of the two particles, A and A', should be similar. The forces tending to rotate grain A come from a finite number of particle contacts while the forces tending to rotate particle A' come from a very large number of molecular collisions of the fluid with the particle. As the size of the mineral grains surrounding particle A in FIGURE III.3 (a) decreases, the number of contact points will increase and the analogy should become more valid.

An oblate particle immersed in a sheared viscous fluid is shown in FIGURE III.3(b). If θ , the angle between the long axis of the mineral grain and the direction of shear, is not zero, it is evident that a couple must act upon the mineral grain and tend to align it with the direction of shear.

The sample in a direct shear test may be considered to be in a state of simple shear. As can be seen in FIGURE III.3 (c), a thin zone should exist in which the degree of particle orientation should be high at large horizontal displacements. This simple approach is confirmed by the theoretical results of Bhattacharyya (1966) and Gay (1966, 1968a and 1968b).

Bhattacharyya (1966) studied the motion of prismatic particle in Couette flow (defined from the velocity components $u = gy$, v and $w = 0$ where u , v and w are velocity components of a viscous fluid along the x , y and z axes and g = velocity gradient). For the situation shown in FIGURE III.4 the particle was found to rotate with the following angular velocity:

$$\frac{d\phi}{dt} = \frac{g}{r_e^2 + 1} (r_e^2 \cos^2 \phi + \sin^2 \phi) \quad (1)$$

where: g = velocity gradient = $\frac{du}{dy}$

r_e = axial length of the grain (length/diameter)

ϕ = angle between the y axis and the projection of the long axis of the prismatic grain on the xy plane.

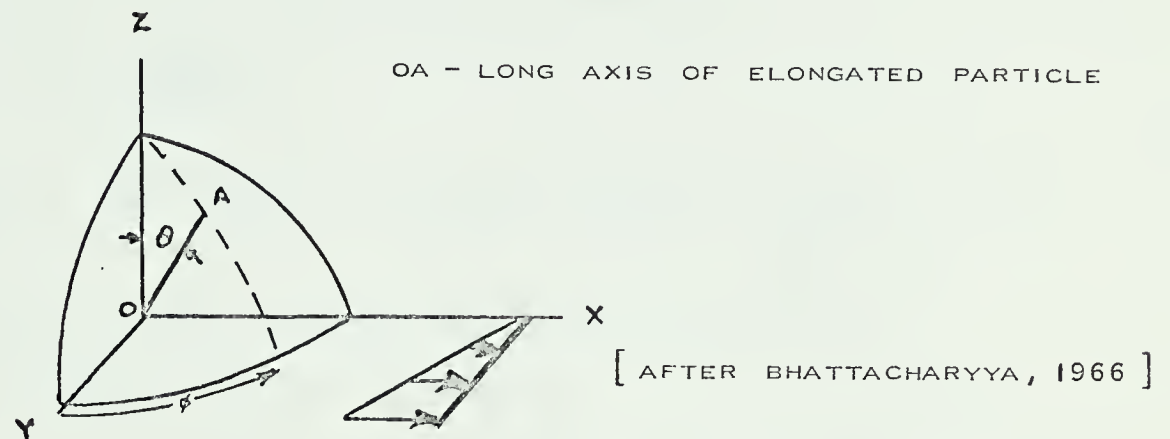


FIGURE III . 4 - PRISMATIC PARTICLE IN COUETTE FLOW

Bhattacharyya also states:

$$\tan \theta = \frac{c r_e}{(r_e^2 \cos^2 \phi + \sin^2 \phi)^{\frac{1}{2}}} \quad (2)$$

where: c = a constant of integration and defines the particle motion. When $c = 0$ the long axis of the particle, oa , remains parallel to the z axis. When $c = \infty$ the axis oa rotates within the xy plane.

From Equation (1) $d\phi/dt$ will be a minimum when $\phi = 90^\circ$. Thus, if $c = \infty$, the majority of the particles will be aligned parallel to the direction of flow.

From Equation (1), at high viscosities, a decrease

in g will decrease the angular velocity $d\phi/dt$. Thus at high viscosities of the surrounding fluid, the particles will tend to spend a long period of time such that ϕ is close to 90° . In this position the shear of the flowing medium will rotate the particle until $\phi = 90^\circ$. Thus the elongate particles will align parallel to the x axis (the direction of flow).

If in (1) $\phi = 0$ the equation reduces to

$$\frac{d\phi}{dt} = g \frac{r_e^2}{r_e + 1} \quad (3)$$

If $\phi = 90^\circ$ Equation 1 reduces to $\frac{d\phi}{dt} = g \frac{1}{r_e^2 + 1}$ (4)

Thus for a very long and thin particle (say for example $r_e = 100$) Equation (3) reduces to

$$\frac{d\phi}{dt} = 100 g \quad (\text{maximum rotation})$$

Equation (4) reduces to $\frac{d\phi}{dt} = 0$ (zero rotation)

Thus Bhattacharyya predicts for large values of axial ratio, the angular velocity of particles oriented parallel with the direction of shear should be zero. Particles not aligned with the direction of shear (or flow) will have an angular velocity tending to align them with the direction of shear. The tendency for particle alignment with the direction of shear can be seen to be a direct function of the axial ratio of the particle.

Gay (1966) considers the motion of particles in a viscous fluid subjected to a pure shear deformation. This state of shear is defined by the equations $u = \dot{\epsilon} x$; $v = -\dot{\epsilon} y$; $w = 0$.

where u , v and w represent the velocity components parallel to the x , y and z axes. Pure shear is an irrotational strain involving shortening of the sample in one direction and extension in the other. The development of this case is not reproduced here but Gay concluded that a elongate particle, immersed in a viscous fluid subjected to pure shear, will rotate towards the direction of elongation of the sample.

For the case of simple shear acting upon a viscous fluid Gay predicts that deformable particles immersed in this fluid "will deform and rotate towards the shearing direction, the amount of deformation and rate of rotation being a function of the viscosity ratio between the particle and its surrounding material." The degree of preferred orientation depends upon the shape of the particles, "the more eccentric the particles, the longer the time they will require to rotate out of parallelism".

Thus under simple shear Gay predicts that granular particles immersed in a viscous fluid rotate towards the direction of shear. A preferred orientation will be set up; however, there will not be perfect orientation because particles rotate out of, as well as into, alignment with the direction of shear. Mason and Manly (1956) studied the motion of thin rods suspended in a sheared viscous fluid. A preferred orientation in the direction of shear was noted.

Thus, if similar particle movement occurs in a

sheared granular mass as in a sheared viscous fluid-solid particle mixture, soil particles with large axial ratios should attain a better degree of particle orientation than particles with a small axial ratio.

3.3 PROPERTIES OF GRANULAR SYSTEMS AS APPLIED TO THE DIRECT SHEAR TEST

Roscoe (1953) approached the problem of the stress distribution in the direct shear sample by use of the theory of plasticity. This analysis showed that tension zones should exist in the sample at low normal loads. Similar results can be obtained by considerations of the static equilibrium of an idealized system of spheres. FIGURE III.5 shows a rhombohedral packing of equal, weightless and rigid spheres in a direct shear box with no normal load acting on the top of the sample.

Shearing force F is applied to the bottom half of the shear box and the reaction $R = F$ acts on the top half. A condition of limiting equilibrium is assumed (motion is impending). It is apparent that movement will occur first along plane A-A and that the area to the left of this plane and to the right of plane B-B are "dead" as no intergranular forces can act in them by simple statics.

Failure can be taken to occur by a spherical particle moving from position one to two in FIGURE III.5 over the sphere in the adjacent row. It can be seen that failure (movement) will occur along plane A-A and B-B, or some parallel plane, before failure will occur in the center of the sample. The sample will dilate as the top platten is free

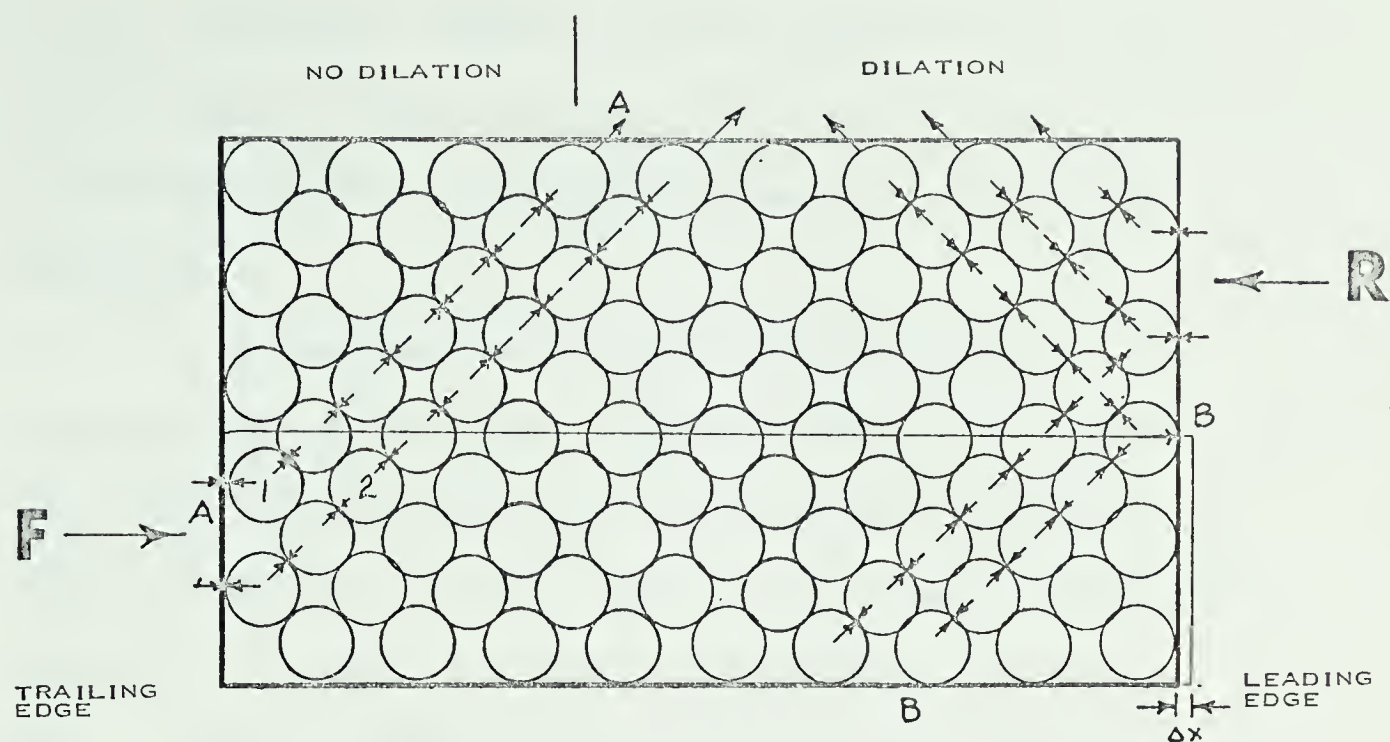


FIGURE III . 5 - EQUAL SPHERES IN RHOMBOHEDRAL PACKING IN DIRECT SHEAR

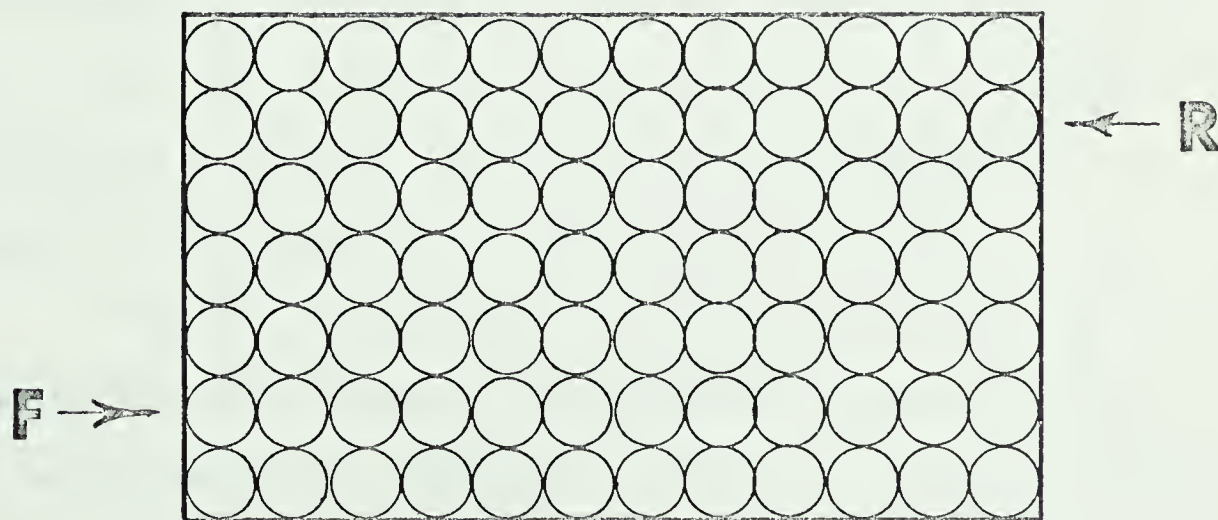


FIGURE III . 6 - CUBIC PACKING IN DIRECT SHEAR

to move vertically. As can be seen in FIGURE III.5, in the initial stages of shear, only a portion of the surface of the sample will dilate. No dilation can occur to the left of plane A-A.

As movement, and dilation spreads throughout the major portion of the sample, a large number of the spheres will enter a cubic packing as is illustrated in FIGURE III.6. The cubic packing is unstable but, as can be seen in the figure, there is no tendency for sample dilation in this packing.

In the initial stage of shear, when motion is impending along A-A or some small movement has occurred, some movement of the bottom half of the shear box, Δx , has occurred. Thus, as is illustrated in FIGURE III.5, the row of spheres on the outside of the sample in the lower right of the box will move to the right as there is no intergranular forces to inhibit their movement. Intergranular forces act along plane B-B. The zone to the right of plane B-B could be said to be in tension although the term "dead" best describes the intergranular stresses in this zone before particle movement occurs.

Farouki and Winterkorn (1964) point out that, in a system of uniform spheres, the rhombohedral packing is the most dense and stable configuration which the constituent spheres can achieve. The cubic packing is unstable as the smallest disturbance will return the spheres to the rhombohedral state. The rhombohedral packing has the highest shearing resistance of all the packings possible.

If equal spheres accumulate on a horizontal surface there will be a strong tendency for a simple rhombohedral arrangement to occur. This packing will propagate upwards to form rhombohedral packing interrupted by regions of cubic and haphazard packing (Farouki and Winterkorn, 1964).

From this evidence it appears likely that a good deal of a sample of cohesionless soil, when placed in a direct shear box, will be arranged in a rhombohedral packing. Vibration or tamping of the sample would increase the percentage of the sample in the rhombohedral packing.

A shearing force is applied and particles move from the rhombohedral to the cubic packing. Sample dilation occurs as is shown in FIGURE III.7 where sphere A moves to position B. Further displacement moves the sphere to C, back into the rhombohedral packing, and a resulting sample contraction occurs.

Failure in the direct shear test progresses from the ends of the box (Hvorslev, 1960). As failure proceeds into the center of the box the number of spheres being forced into the cubic packing (with resultant sample dilation) is larger than the number of spheres returning to the rhombohedral packing (with an accompanying volume decrease) and the sample dilates. Ultimately, along the center of the sample, an equilibrium is reached where the number of spheres going from the rhombohedral to the cubic packing equals the number of spheres going from the cubic to the

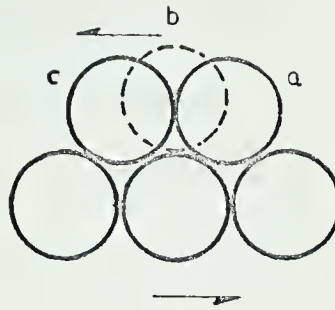


FIGURE III.7 - MOVEMENT OF PERFECT SPHERES UNDER SIMPLE SHEAR

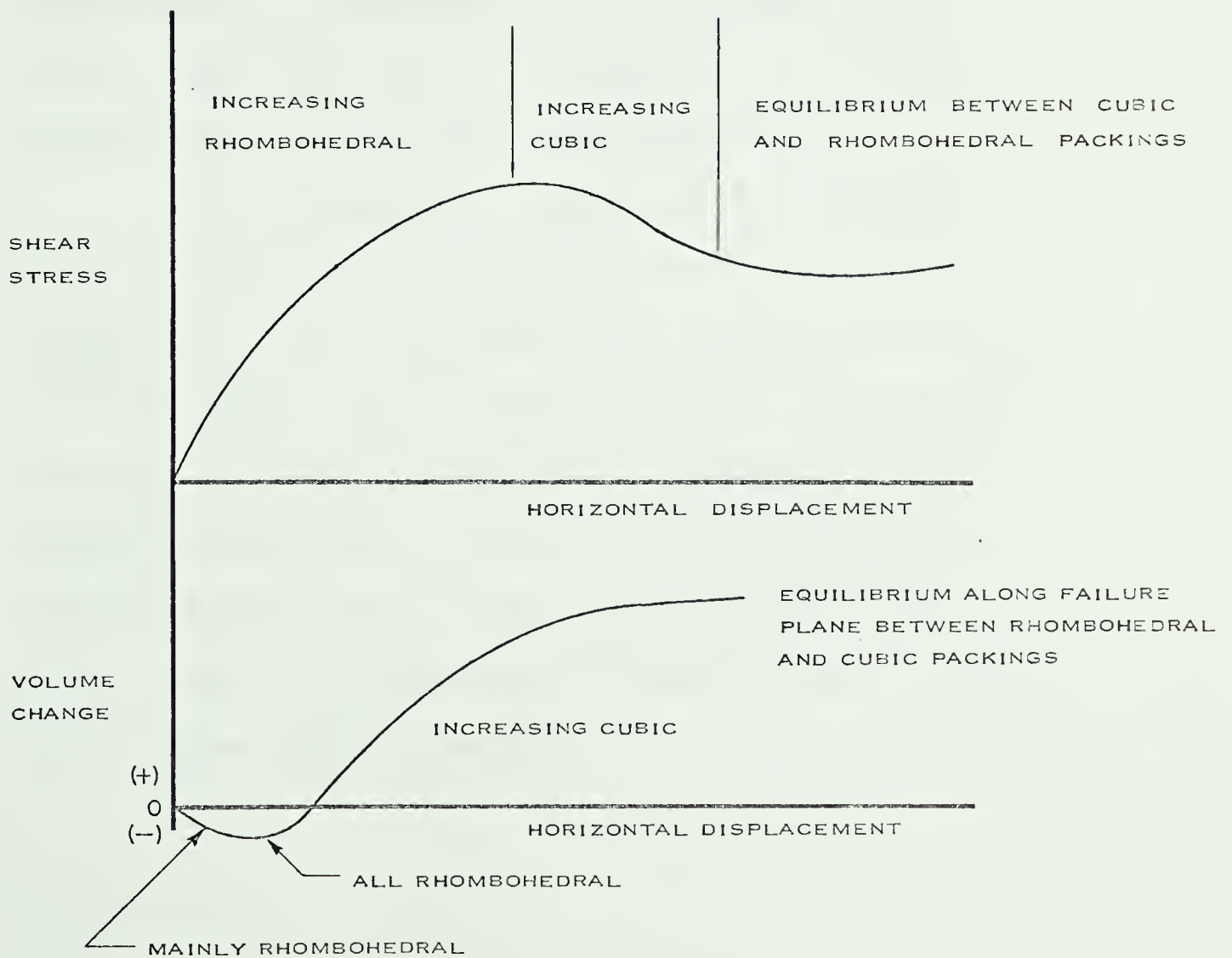


FIGURE III. 8 - TYPICAL DIRECT SHEAR TEST RESULTS FOR DENSE GRANULAR MEDIA

rhombohedral packing.

Although this model is strictly applicable only to equal spheres it should hold true, in a general manner, for packing of non-spherical grains.

The results of a typical direct shear test upon dense granular soil is given in FIGURE III.8. In the light of the discussion on the mechanics of granular soils one would expect to find an increase in the stress-displacement curve (increased shear strength) accompanied by a decrease in the volume of the sand. The residual strength of the sample should be reached when no further sample dilation occurs. Test results show this is not the case and this anomaly will be further discussed in Chapter VI.

It is of interest to note that characteristic small volume decrease at the start of shear as shown in FIGURE III.8. This corresponds to the transition of most of the particles into rhombohedral packing. The dilation following should occur initially in the portion of the sample between planes A-A and B-B as shown in FIGURE III.5. Dilation should occur only over a portion of the surface of the sample. The experimental evidence tending to confirm this is given later in Chapter V.

CHAPTER IV

EXPERIMENTAL PROGRAMME

4.1 GENERAL

The aim of this thesis was to study the behaviour of granular materials during the direct shear test. Samples of crushed calcite, after being sheared to a predetermined displacement, were carbowaxed. The samples were prepared in such a manner that the relative positions of the calcite grains could be seen. Other direct shear tests were carried out in order to study the mechanism of shear in the direct shear box and the distribution of stress within the shear box.

4.2 MATERIAL USED

Preliminary carbowaxing and grinding of naturally occurring quartz sands showed that Carbowax 6000 was insufficiently strong to hold the quartz grains during grinding. Plucking of the grains resulted and use of quartz sands had to be abandoned.

It was decided to form an artificial sand by crushing and sieving a soft rock. Calcite was decided upon and its properties are shown in TABLE IV.1.

Samples of calcite were obtained from the Geology Department, University of Alberta and from the Alberta Granite

TABLE IV.1

PROPERTIES OF CALCITE¹

Specific Gravity	2.71
Moh's Hardness Number	3
Lustre	Vitreous
Chemical Composition	CaCO ₃ with substitution of Mg or Fe for Ca
Colour	Colorless or white when pure, varying with substitution or inclusion
<hr/>	
(1) From Berry and Mason (1959)	

and Marble and Stone Company Limited of Edmonton. Both samples had specific gravities of 2.71 although they had slightly differing colours due to differing inclusions. The calcite obtained from the Geology Department had a milky yellow tinge and was more opaque than the other sample which was white.

The samples fractured easily into small pieces and were then fed into a Massco mechanical crusher. The resulting calcite sand was sieved and stored in plastic bags. Material coarser than the number four U.S. standard sieve size was re-crushed (0.187 inch). The sand was stored corresponding to material retained on the number ten U. S. standard sieve (0.0787 inch), the number twenty (0.0331 inch) and the number forty (0.0165 inch).

Using this material specimens were mixed to desired grain size curves by weighing out the required components to ± 0.02 gram. Direct shear tests were conducted upon the resulting calcite sands.

4.3 DIRECT SHEAR TESTING AND SECTION PREPARATION

Samples were tested in the hand operated direct shear machine shown in APPENDIX B. The shear box was bronze with an internal length and width of 2.60 inches and a depth of 2.00 inches.

The testing programme consisted of five test series. These are shown in TABLE IV.2.

TABLE IV.2

DIRECT SHEAR TESTING PROGRAMME

SERIES	NO. OF SAMPLES	AIM
A	16	EXPLORATORY AND TO STUDY THE EFFECT OF HORIZONTAL DISPLACEMENT.
B	15	EFFECT OF DENSITY
C	6	EFFECT OF GRAIN SIZE DISTRIBUTION
D	3	EFFECT OF NORMAL LOAD
E	5	RECAPITULATE C AND D
F	6	STUDY PARTICLE CRUSHING AND SURFACE MOVEMENT (NOT SECTIONED)

The sample of calcite sand, mixed to the desired grading, was placed in the direct shear box; care being

taken to achieve a uniform density in each test series. The volume of each sample was measured to obtain the initial void ratio. The samples were sheared at the rate of 0.008 inches per second to preselected horizontal displacements with the shearing force being measured by a proving ring and the vertical movement of the load head being measured by a dial gauge (1 division = 0.0001 inch).

The shear box was removed from the machine and placed in an oven at 160°F. Care was taken to reduce sample disturbance to a minimum during the transfer. After fifteen minutes the shear box was removed from the oven and previously melted Carbowax 6000 was poured over the top of the sample and allowed to percolate down through it. Full details of experimental procedure are given in APPENDIX B.

Upon cooling, the sides of the shear box were gently heated with a propane torch. The sample was extruded and allowed to cool. It was then sliced into four longitudinal slabs using a standard table saw.

The resulting slabs of the sample yielded a view of a vertical section oriented parallel to the direction of shear. Selected slabs of each sample were ground on a lapping wheel to remove any disturbance which might have been caused by the slicing of the sample. During the grinding numbers 220 and 600 silicon carbide abrasive powder were used with varsol serving as a lubricant. This stage of the process is illustrated by PLATE 2 included in APPENDIX B. The polished

face of the slab was mounted on a glass microscope slide. The sample was then ground down to a thickness of about 0.01 inch on the lapping wheel. The glass slide was then marked with a glass etcher for future identification. Each slide was marked with the Test Series (A to E), the number of the sample and the number of the slab. Thus the marking A-11-3 means that the slide under study was the third slide prepared from Sample 11 in Test Series A.

The procedure for the preparation of thin sections, in general, followed that of Mitchell (1955). As the resulting thickness of the sample was considerably greater than the 30 to 40 microns used by Mitchell, the finished samples when ready for study are referred to as "sections" rather than "thin sections". Full details of experimental procedure including section preparation are given in APPENDIX B.

TABLE V.1 contains details of the testing programme. In general, one section was prepared from each sample but, in some cases, several were made to study the effects of using different adhesives or to study "edge effects".

Three different adhesives were used to fix the polished face of the specimen to the glass microscope slide. These were Permunt, Lakeside 70C and Epoxy 220. Details on their effectiveness and uses are also contained in APPENDIX B.

The finished sections were cleaned with varsol and placed in a stand as shown in FIGURE IV.1. The sections were lighted from the back and photographed with a Hasselblad camera using a number 55 extension tube and an 80 millimeter

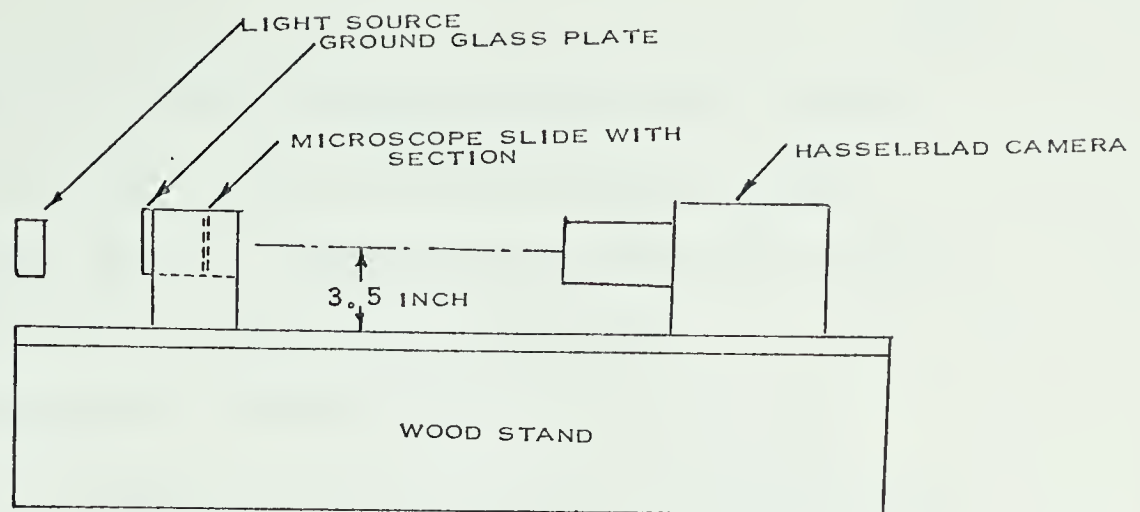


FIGURE IV. 1 - DIAGRAMMATIC SKETCH OF PHOTOGRAPHIC STAND

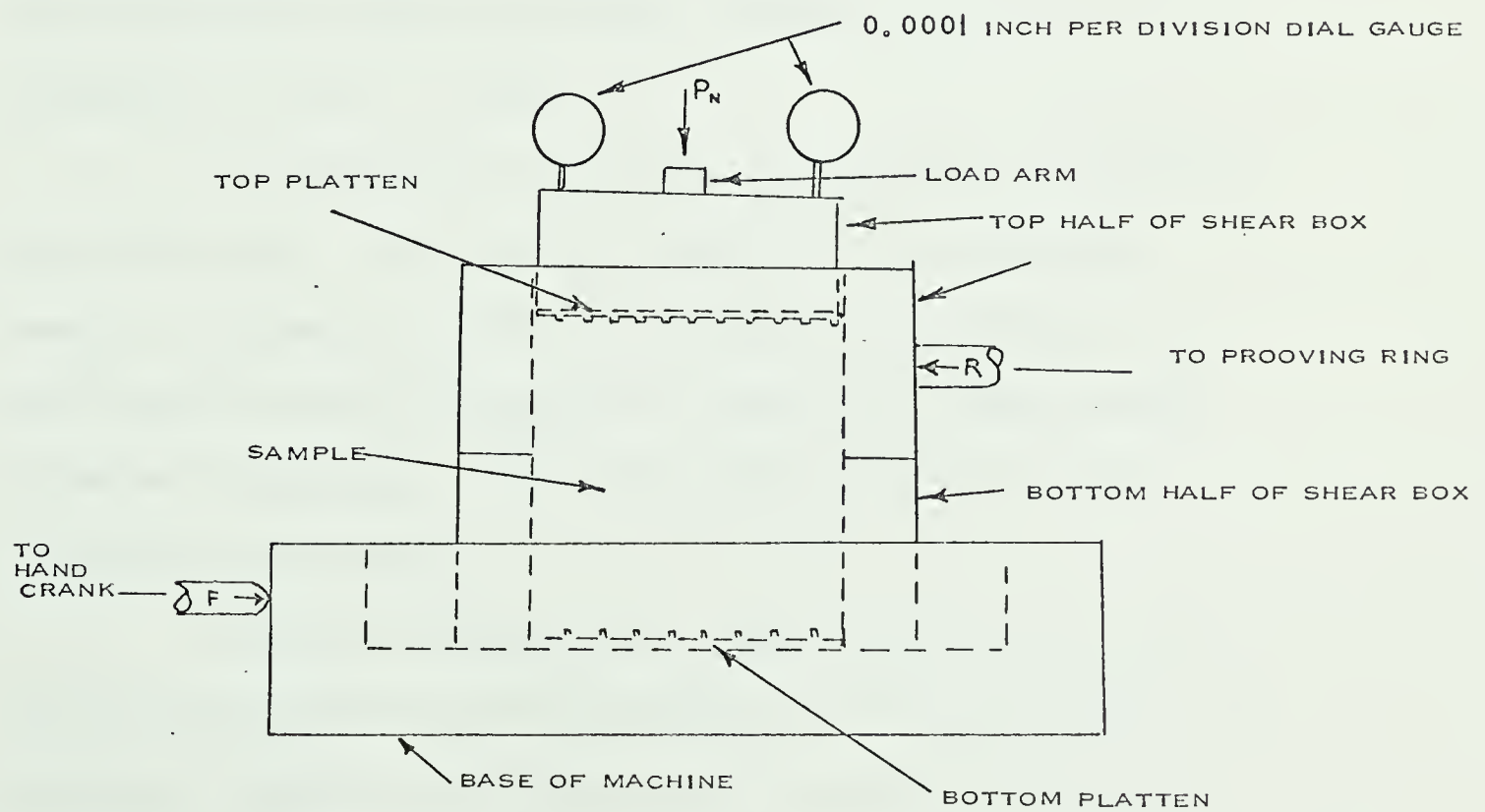


FIGURE IV. 2 - DIAGRAM OF DIRECT SHEAR MACHINE

lens. Negatives two and one quarter inches square resulted. When enlarged to eight and one half by eleven inch size, a magnification of about 3.5 resulted which showed the calcite grains with sufficient clarity for this work.

Photographs of selected sections are given in APPENDIX C. The results of interpretation of these sections are given in Chapter V.

4.4 ADDITIONAL TESTING

In order to study the behaviour of granular materials in the direct shear box during shear, Test Series F was carried out. Direct shear tests were conducted on calcite sands using two dial gauges to measure differential movement of the ends of the sample surface. The test apparatus is shown in FIGURE IV.2.

All samples were sheared to a horizontal displacement of 0.400 inches. The vertical movement of the ends of the sample, as measured by the two dial gauges, was recorded. At the end of the test the sample was removed and sieved through a number twenty sieve. The amount passing (due to particle crushing) was measured.

The tests conducted at a normal pressure of 0.1 p.s.i. had the load head removed and weights set directly upon the top platten. The ends of the dial gauges were also set directly upon this platten.

Several direct shear tests were carried out at zero normal load with the top of the calcite sample visible. The

depth from the top of the shear box to the surface of the calcite was measured at zero displacement and at varying horizontal displacements. Results are given in Chapter V.

The effect of time upon the properties of liquid carbowax was observed. Twelve dishes of flaked carbowax were placed in an oven at 160°F. They were removed after varying lengths of time and 20 ml. of the liquid carbowax was poured in a glass beaker. The length of time required for solidification of the carbowax was noted.

4.5 ANALYSIS OF TESTING PROCEDURE

It is felt by the author that the finished sample sections give an accurate picture of the orientation of the particles at the end of horizontal displacement. It is not felt that an accurate picture of voids and particle packings is given for reasons discussed fully in Chapter VI.

During the carbowaxing every effort was made to reduce the amount of sample disturbance to a minimum. Solidification of the carbowax is accompanied by small volume changes, as has been previously discussed in Chapter II and thus the particle orientation should not be affected.

Grinding of the samples prior to mounting removes any surface disturbance due to slicing of the sample. No change on particle orientation occurred in the final grinding process. It could be clearly seen that particles were merely being abraded and not rotated.

CHAPTER V

PRESENTATION OF RESULTS

5.1 GENERAL

The negatives of the sections photographed were expanded to eight and one-half by eleven inch size giving an average magnification of about 3.5. Photographs of selected sections are given in APPENDIX C. Observations from the study of these slides are given in this chapter and are correlated with test data obtained from the direct shear test.

5.2 SHEAR STRENGTH CHARACTERISTICS OF CALCITE SANDS

The data obtained from the direct shear tests are summarized in TABLE V.1. As can be seen from the initial void ratios, samples in Test Series A, C, D and E were not in a loose state when sheared. They were not, however, in a loose state as dilation occurred during shear. A typical stress-displacement curve and volume change curve is shown in FIGURE V.1 and the plots of the test data for the remaining tests are shown in APPENDIX D. Thus the samples in Test Series A, C, D and E could be said to be of medium density while samples in Test Series B were in a dense packing.

The variation in grading of the samples had some effect on the peak value of the stress-displacement curve. Taking the peak stress as representing failure in the sample,

TABLE V.1

SUMMARY OF DIRECT SHEAR TESTING ON CALCITE SANDS

SAMPLE NUMBER	PEAK STRENGTH (P.S.I.)	HORIZONTAL DISPLACEMENT (INCH)	NORMAL PRESSURE (P.S.I.)	SAMPLE GRADING	INITIAL VOID RATIO
A-1	--	0.200	6.03	WELL	--
2	6.0	0.272	6.03	GRADED	--
4	6.8	0.272	6.03	#10-#40	--
5	5.8	0.280	6.03	#10-#40	1.18
6	5.7	0.120	6.03	#10-#40	1.13
7	--	0	6.03	#10-#40	1.08
8	--	0	6.03	#10-#40	--
9	--	0	6.03	#10-#40	--
10	--	0.400	6.03	#10-#20	0.92
11	6.5	0.400	6.03	#10-#20	1.07
12	--	0.096	6.03	#10-#20	0.98
13	6.3	0.200	6.03	#10-#20	0.86
14	0	0	6.03	#10-#20	0.96
15	0	0	6.02	#10-#20	1.10
16	0	0	1.59	#10-#20	1.04
B-1	11.0	0.280	6.03	#10-#20	0.65
2	9.2	0.400	6.03	#10-#20	0.80
3	6.9	0.200	6.03	#10-#20	0.67
4	7.0	0.120	6.03	#10-#20	0.78
5	0	0	6.03	#10-#20	0.72
C-1	6.9	0.400	6.03	#4-#10	1.00
2	6.7	0.400	6.03	#4-#40W.G.	0.95
3	6.2	0.400	6.03	#4-#40W.G.	0.98
4	6.3	0.400	6.03	#4-#40G.G.	0.98
5	6.6	0.400	6.03	#4-#40G.G.	1.05
6	7.3	0.400	6.03	#4-#20G.G.	0.96
D-1	10.1	0.400	10.5	#4-#40W.G.	0.91
2	16.4	0.400	17.1	#4-#40W.G.	0.86
3	25.1	0.400	23.8	#4-#40W.G.	0.84
E-1	6.2	0.400	6.03	#4-#20W.G.	0.86
2	7.0	0.400	6.03	#4-#20W.G.	0.90
3	0	0	6.03	#4-#20W.G.	0.94
4	11.5	0.400	10.5	#4-#20W.G.	0.90
5	0	0	10.5	#4-#20W.G.	0.94

NOTE: G.G. refers to gap-graded samples
W.G. refers to well-graded samples

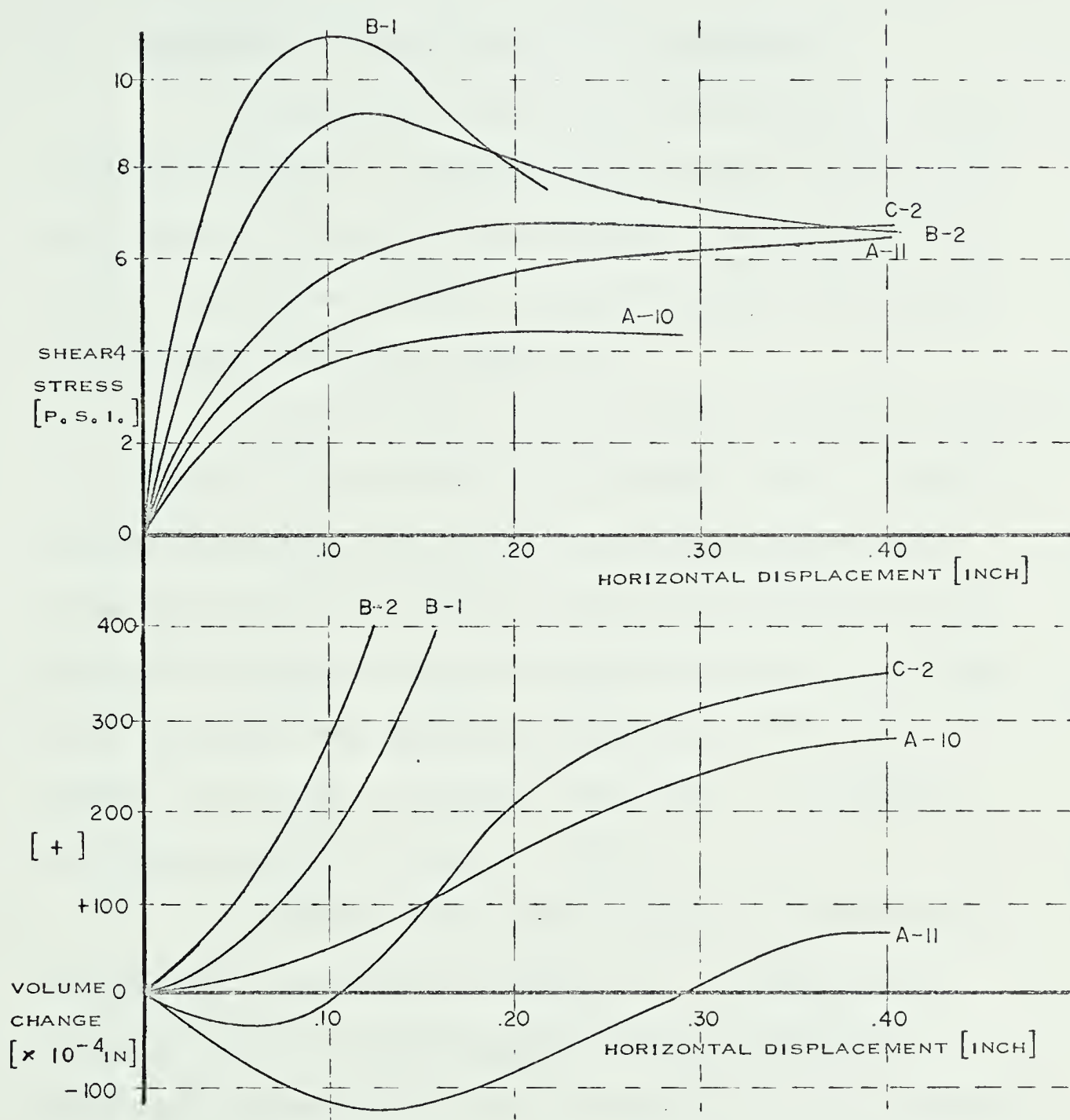


FIGURE V.1 - PLOT OF TYPICAL DIRECT SHEAR RESULTS
FROM TEST SERIES A, B AND C

a Mohr rupture line is shown in FIGURE V.2 for samples with the same grading. Thus crushed calcite, with the grading specified in FIGURE V.2 has an angle of internal friction of 46° .

5.3 THE EFFECT OF SHEAR UPON PARTICLE ORIENTATION

The photograph of each section was carefully studied. In the sections where grain boundaries could be distinguished, the long axes of elongated particles were marked after study of the original section with a magnifying glass. The results of this study are given in TABLE V.2.

It was difficult to assess trends in particle orientation from a visual inspection of each photograph. Thus a number of sections from Test Series A were chosen for further study. From the center third of five sections, the departure of the long axis of each oblate particle from the direction of shear (the horizontal) was assessed as being low (0° to 30°), medium (30° to 60°) or high (60° to 90°). The results for samples studied are given in TABLE V.2.

The results of the above analysis for Test Series A are plotted in polar form in FIGURE V.3. A study of this diagram, and the accompanying photographs in APPENDIX C, shows some preferred horizontal orientation at zero displacement. Increasing displacement shows a gradual rotation of the particles through the vertical (to the direction of movement) until a definite preferred orientation is set up paralleling the direction of movement. Section A-11, at a horizontal displacement of 0.400 inch, shows a much greater degree of

TABLE V.2

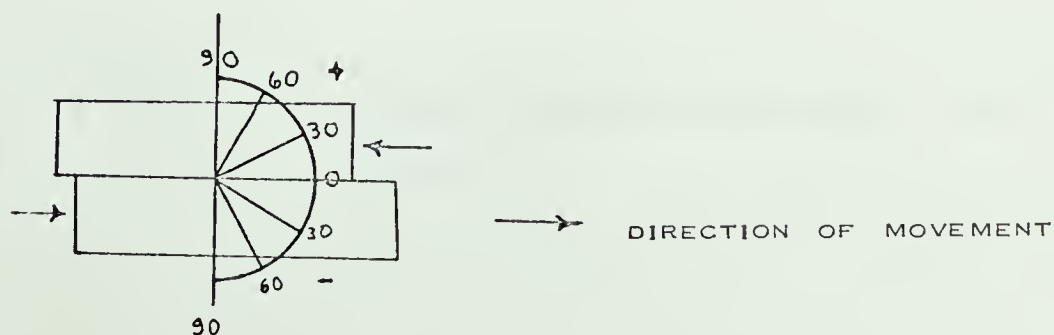
PARTICLE ORIENTATION OBSERVED

SECTION	HORIZONTAL DISPLACEMENT (INCH)	VOID RATIO	NORMAL STRESS (P.S.I.)	NUMBER OF PARTICLES					
				(+) ORIENTATION ¹			(-) ORIENTATION ²		
				LOW	MEDIUM	HIGH	LOW	MEDIUM	HIGH
A-1	0.200	-	6.03	21	11	16	29	27	15
5	0.280	1.18	6.03	12	12	10	14	8	11
6	0.360	1.13	6.03	15	2	6	12	3	1
7	0.120	1.08	6.03	6	2	9	9	6	9
8	0.000	-	6.03	8	6	12	9	5	9
11-2	0.400	1.07	6.03	33	21	19	21	11	13
12	0.096	0.98	6.03	21	15	16	18	11	21
14	0.000	0.96	6.03	18	13	14	18	17	7
16	0.000	1.04	6.03	25	17	17	20	14	24
B-1	0.280	0.65	6.03	28	21	17	23	13	14
2	0.400	0.80	6.03	22	22	11	13	10	10
3	0.200	0.67	6.03	25	16	24	35	10	20
C-1-1	0.400	1.00	6.03	21	6	8	18	16	18
1-3	0.400	1.00	6.03	29	14	10	26	15	8
6*	0.400	0.96	6.03	13	15	14	14	10	6
D-1*	0.400	0.91	6.03	15	8	10	14	12	5
2*	0.400	0.86	17.1	17	15	9	16	10	6
3*	0.400	0.84	23.8	14	11	6	24	10	4
E-1-1	0.400	0.86	6.03	20	9	10	11	7	11
1-2	0.400	0.96	6.03	18	8	7	11	10	14
2-2	0.400	0.90	6.03	25	11	17	33	20	13
3-1	0.000	0.94	6.03	23	19	11	21	15	13
5-1	0.000	0.92	10.5	15	16	13	18	16	14

1. Orientation is measured from the direction of movement (i.e. the horizontal) (-) clockwise, (+) counterclockwise.

2. Orientations are low 0° - 30° , medium 30° - 60° , high 60° - 90° .

* Denotes difficulty in distinguishing grains, results are only approximate.



NOTE - ALL SAMPLES HAVE THE SAME GRADING-33.3% NO. 4 - NO. 10,
33.3 % NO. 10 - NO. 20 , 33.3% NO. 20 - NO. 40

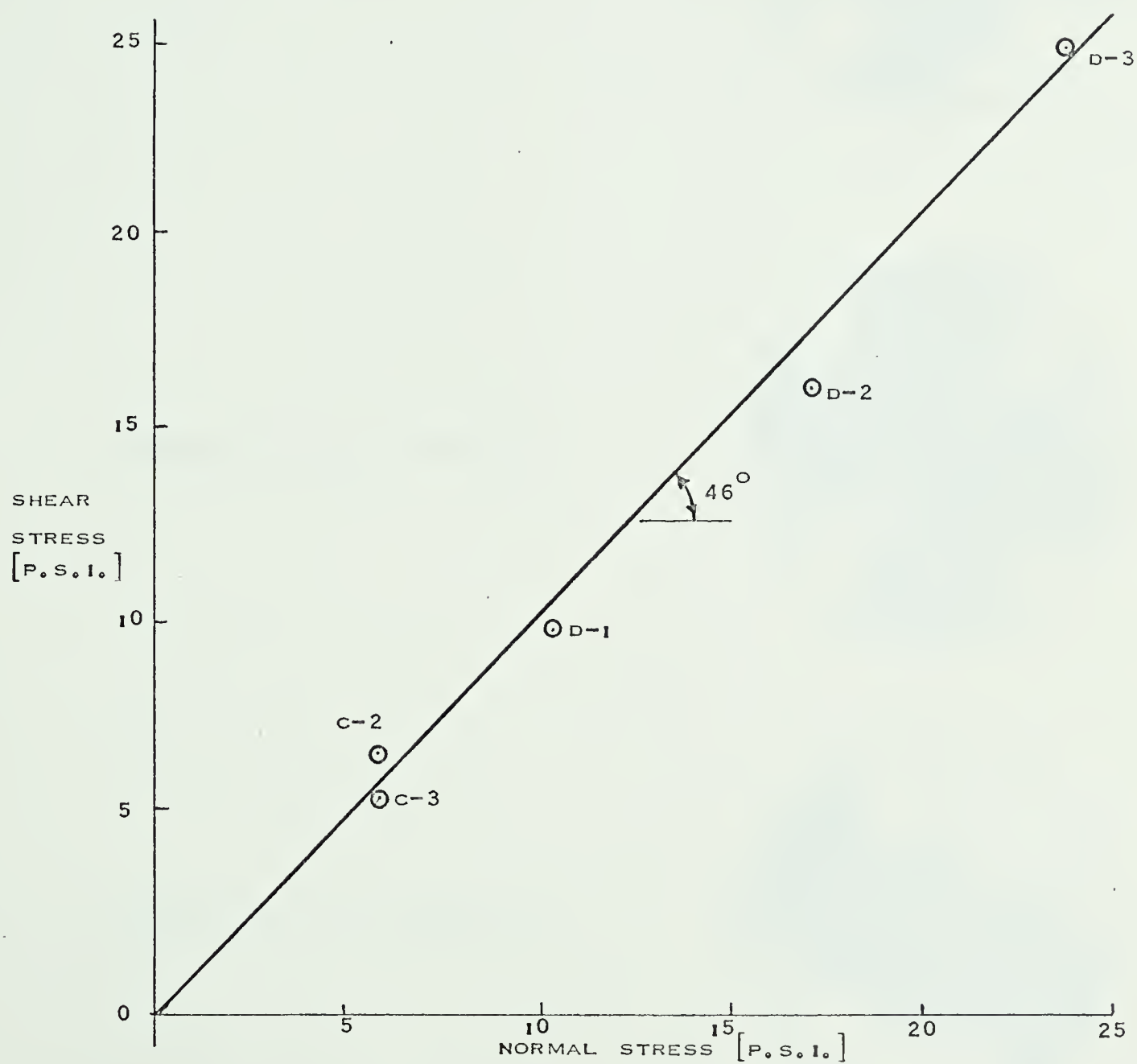
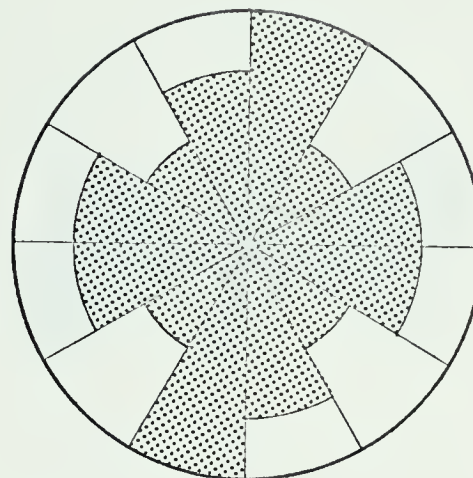
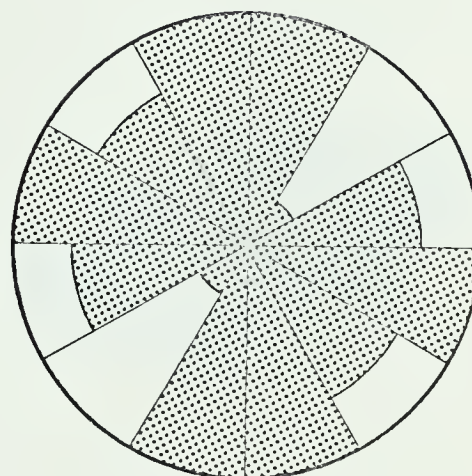


FIGURE V.2 - MOHR-COULOMB RUPTURE LINE FOR CALCITE SANDS.

SAMPLE A-8, HORIZONTAL DISPLACEMENT
[H.D.] -0.00 INCH



SAMPLE A-7, H.D. -0.120 INCH



SAMPLE A-1, H.D. -0.200 INCH

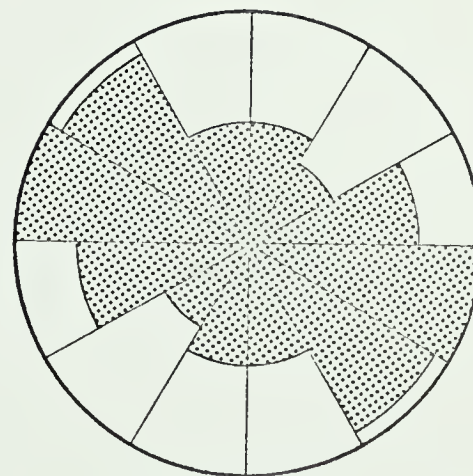
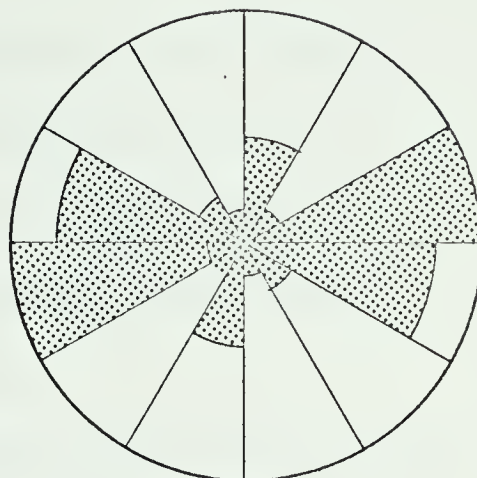
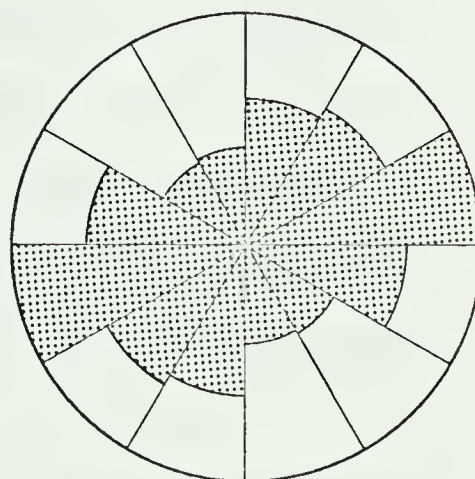


FIGURE V.3 - DIRECTIONAL ROSETTES SHOWING ORIENTATION
OF THE LONG AXES OF CALCITE GRAINS [SERIES A]

SAMPLE A-5, H.D. - 0.280 INCH



SAMPLE A-II, H.D. - 0.400 INCH



horizontal orientation than section A-8 which was taken from an unsheared sample.

It must be kept in mind that these results shown in FIGURE V.3, reveal only trends; minor variations will occur due to errors introduced by the relatively small number of particles which were counted, Other reasons for introduction of errors into this method of analysis are:

1. orientations of the particles were placed in the three categories previously mentioned. Thus quantities may not be exact.

2. only the center third of each section was analyzed. This was done to include the center portion of the sample, theoretically in simple shear, and to exclude edge effects. Analysis of a zone of this thickness could have included material not in a state of simple shear.

Although some of the samples of Series A have been sheared past any peak which might exist, the results shown in FIGURE V.3 clearly indicate that particle orientation in the center of the sample is not perfect even at large displacements (0.400 inch). A study of the photographs of Test Series A show no failure plane or zone exists. Preferred orientation exists in the upper and lower portions of the sample, it is not confined to the zone near the plane of separation of the two halves of the shear boxes.

A study of the photographs of samples A-1 and A-7 (page C-1 and C-4) indicates that strong preferred particle

orientation (and hence particle movement or sample failure) exists along two arcs as illustrated in FIGURE V.4. It is of interest to note that, in both of the sections A-1 and A-7 taken at low displacements, most of the grains in the center of the sample are at nearly right angles to the direction of movement. This point is further discussed in Chapter VI.

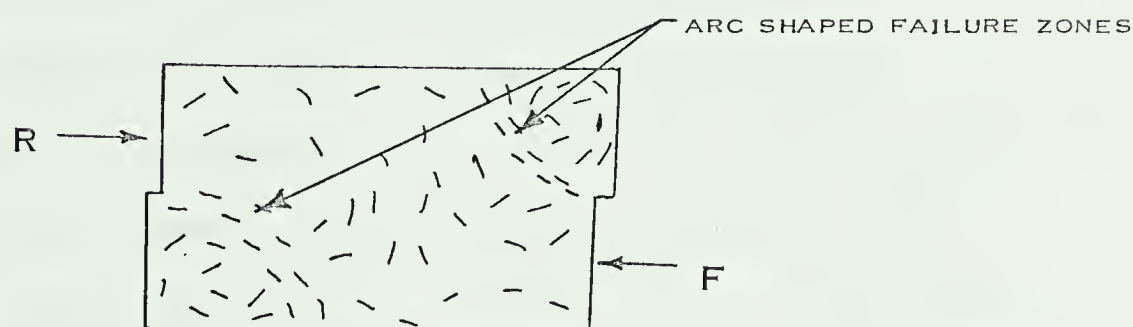


FIGURE V.4 - SKETCH OF PARTICLE ORIENTATION AT LOW DISPLACEMENTS FROM SAMPLES A-1 AND A - 7

As the horizontal displacement increases the orientation in the center of the sample increases and the two "arc failures" gradually become more disordered and tend to disappear. A study of samples of medium density sheared to a large horizontal displacement (A-11, C-1 to 6, D-1 to 3, E-1, E-2 and E-4) shows a preferred horizontal orientation along the center of the sample but little else.

These samples, at high displacement, also display a circular pattern in the upper trailing and lower leading portions of the sample. FIGURE V.5 illustrates this observation. It was found necessary to ink in the direction of the long axis of

each grain in order to observe trends in particle orientation. Visual observation of the photograph yielded little information until a large number of samples had been studied.

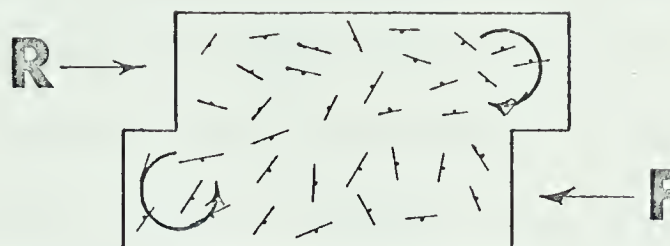
It would appear that during the shearing phase of a direct shear test, the solid grains behave like a fluid. The patterns illustrated in FIGURE V.5 are similar to the vortices which would form if a viscous fluid were sheared.

Most of the sections in Test Series B, C and D proved difficult to analyze. The grains of the calcite used in these series (from the Alberta Marble, Granite and Stone Co. Ltd.) were crushed too small in size (#10 and smaller) and as they contained few impurities and were therefore practically colourless, these grains failed to show their boundaries as well as the calcite obtained from the Geology Department. Only twenty-three of these sections were analyzed and some of the results must be considered approximate as it proved difficult to judge the position of each grain.

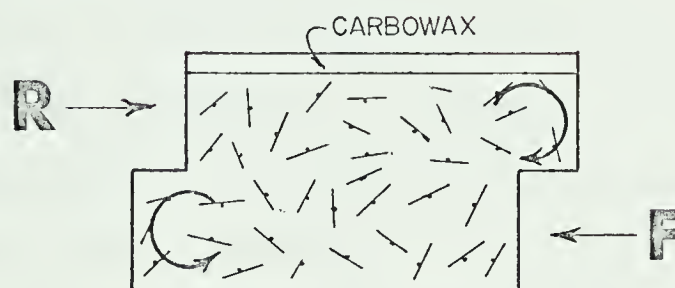
5.4 EFFECT OF VARIABLES ON THE DEGREE OF PARTICLE ORIENTATION

The effect of a number of variables on the resultant degree of particle orientation was studied. The variables studied were density (Test Series B), grading (Test Series C) and normal load (Test Series D). The resulting sections of these series were not, generally, of as good a quality as the initial test series (APPENDIX C). The variables studied yielded the following results.

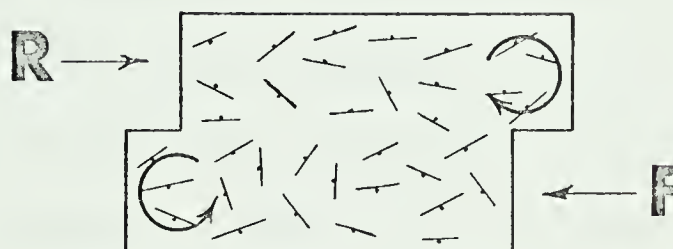
SECTION B-1



SECTION C-1-3



SECTION E 1-2



SECTION D 1-1

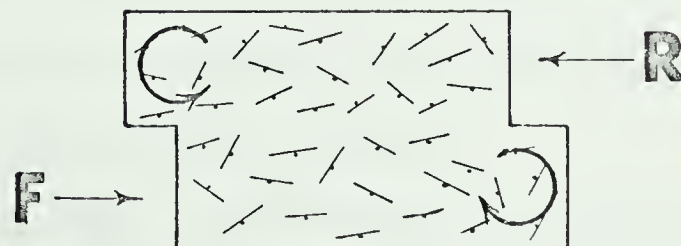


FIGURE V. 5 - SKETCH OF PARTICLE ORIENTATION IN SAMPLES
WITH LARGE HORIZONTAL DISPLACEMENTS

- (a) EFFECT OF DENSITY: Samples in Series B were vibrated to produce a much denser sample than was used for the other series. Studying the photographs of Series B and TABLE V.2, the degree of particle orientation for a given horizontal displacement appears to be less than in the looser test series.

In sections B-1 and B-3 (page C-13 and C-14) there appears to be the beginning of a horizontal failure surface corresponding to that which should be imposed by the geometry of the shear box. The failure plane appears to be a very narrow band of oriented particles. However, it is not well defined.

- (b) EFFECT OF SAMPLE GRADATION: Test Series C used a series of different particle grading to study the effect of this variable upon the degree of particle orientation resulting. A study of the sections in this series shows that no evident trends are apparent. Most of the samples contained too high a percentage of small grains (smaller than #20 sieve size - 0.0331 inch) to enable the grain boundaries to be readily apparent at the magnification (about 3.5) used.
- (c) EFFECT OF NORMAL LOAD: In Test Series D the normal load on the sample was varied. Although the results are not conclusive from a study of only three samples, there does seem to be an increasing preferred horizontal orientation with increasing normal pressure.

5.5 EFFECT OF DETAILS IN EXPERIMENTAL TECHNIQUE

Details in experimental technique were found to have an important effect upon the quality of the resulting sections. Sections in which the calcite grains were visible led to clear and unmistakable results. A study of sections of poor quality led to ambiguous and uncertain results. Thus, use of the proper experimental technique was found to be most important in this work.

The impurities in the calcite used had an important effect upon the ease with which the grains in the resulting section could be distinguished. The calcite provided by the Geology Department had a milky-yellow tinge and was considerable more opaque than that obtained from the Alberta Marble Granite and Stone Company Ltd. The former calcite yielded excellent sections, especially when used with a clear cement (Permunt or Epoxy 220). Sections A-7, C-1-1 and E-1-1 (page C-4, C-17 and C-27) are good examples of sections where the details of particle orientation are clearly visible. However, use of the latter calcite combined with a clear cement produced a section where grain boundaries are indistinct. Section E-4-1 (page C-32) is a good example of the situation.

Lakeside 70C produced a yellow cement which was more opaque than either Permunt or Epoxy 220. If the section was not ground sufficiently thin, a very dark section resulted in which grain boundaries were difficult to locate. Samples A-12 and B-3 (page C-9 and C-15) provide good examples of this.

However, if the section is ground sufficiently thin the grains of calcite are easily seen. Care must be taken, however, not to grind the section excessively and remove all of the calcite. Section A-14 illustrates both of these points.

It was observed that the length of time carbowax had been molten in an oven had a marked effect on the properties of this material. Sample D-1 (page C-24) was carbowaxed with liquid carbowax which had been in the oven for about one month. This carbowax, when solid, proved softer than freshly melted carbowax. In the grinding of section D-1 it was noted that the carbowax held the silicon carbide powder instead of being abraded by it. Thus the carbowax portions of section D-1 appear black due to inclusions of silicon carbide. The calcite grains appear white.

Samples of flaked carbowax were placed in a constant temperature oven which was set at 160°F. Samples of liquid carbowax were removed periodically and allowed to solidify at room temperature. The behaviour of these samples is given in TABLE V.4.

As can be seen from the table, the age of the liquid carbowax has little effect on the time for the formation of particles of solid carbowax upon the surface of the liquid. There is, however, a considerable increase in the length of time required for the wax to solidify. It can also be seen that the solidified carbowax becomes softer and more malleable with age. Thus prolonged heating, in the liquid phase, leads to a change in the physical properties of Carbowax 6000.

TABLE V.3

EFFECT OF AGE UPON THE PROPERTIES OF LIQUID CARBOWAX 6000

DATE	TIME OF REMOVAL	TIME IN LIQUID STATE(HRS.)	OVEN TEMP. °F.	ROOM TEMP. °C.	FIRST SOLIDS ON SURFACE (MIN.)	TIME TO SOLIDIFY (MIN.)	COMMENTS
28 June	11:15	1	168	19.8	11	34	Hard & solid
30 June	11:25	49	172	21.4	10	35	--
2 July	8:32	93	172	20.8	11	37	--
7 July	14:25	219	174	21.1	11	42	Somewhat softer
15 July	11:36	408	181	22.2	11	--	--
16 July	20:43	443	182	21.2	11	44	--
24 July	8:35	621	183	20.4	11	46	Solid but softer (greasy feel)

NOTE: (1) Flaked Carbowax 6000 placed in twelve bowls on June 28, 1968, at 10:15. The bowls were placed in an oven set at 160°F. and the carbowax melted.

(2) On removal, twenty milliliters of liquid carbowax was poured into a glass beaker at room temperature. The time for the first solid specks to appear on the surface was noted as was the time for complete solidification.

5.6 SAMPLE SURFACE MOVEMENT AND SAMPLE CRUSHING IN THE DIRECT SHEAR TEST

Two direct shear tests were carried out to observe the movement of the upper surface of the crushed calcite sample. Details of the tests are given in TABLE V.4. It can be seen from this table that the amount of sample crushing

TABLE V.4

DIRECT SHEAR TESTS TO DETERMINE SAMPLE CRUSHING

TEST	FINAL DISPLACEMENT (INCH)	NORMAL PRESSURE (P.S.I.)	WEIGHT SAMPLE (GRAM)	SAMPLE GRADING	INITIAL VOID RATIO	WEIGHT PASSING #20 AT END (GRAM)
F4	0.152	6.03	210.02	#4 - #20	0.98	1.2
F5	0.400	.10	210.02	#4 - #20	0.99	0.7

which occurs is negligible at small normal pressures.

Two dial gauges were placed upon the ends of the load head of the sample in Test F-4 and upon the top platten in Test F-5. The apparatus used is illustrated in FIGURE V.6. Portions of the sample are designated "leading" or "trailing" and are illustrated in this figure.

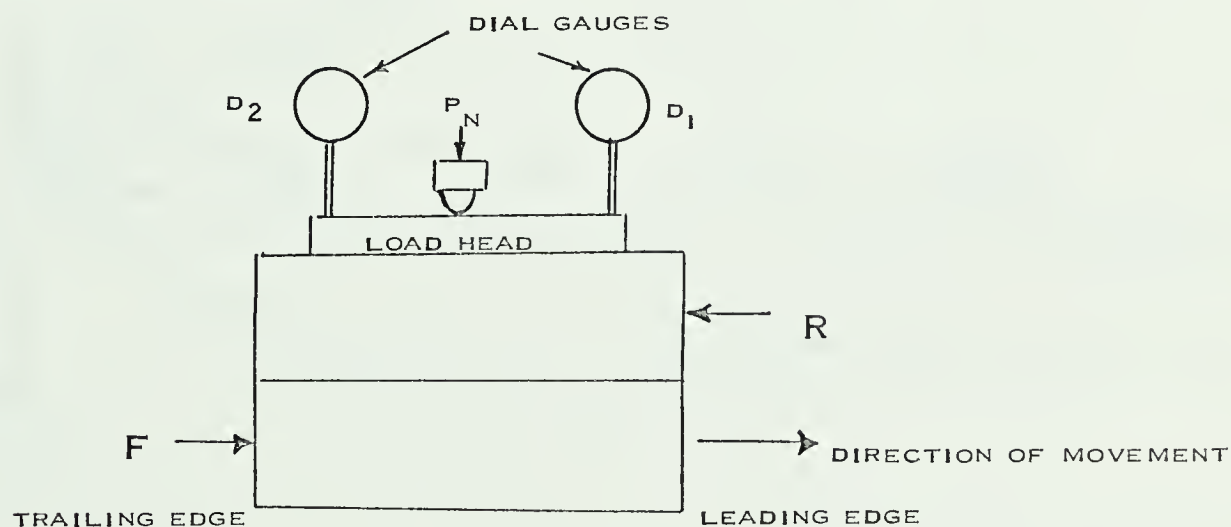
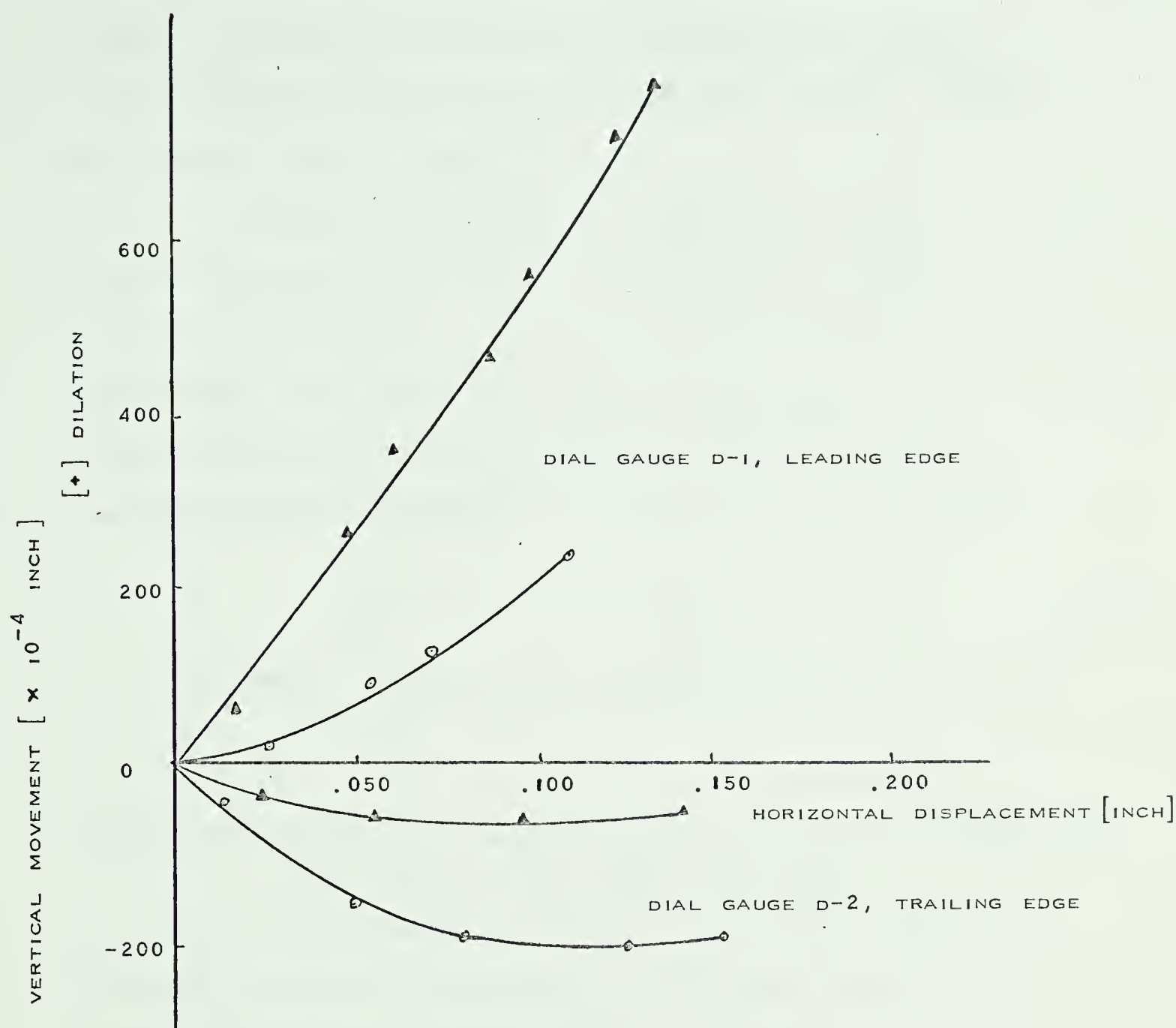


FIGURE V. 6 - DIAGRAM SHOWING APPARATUS TO MEASURE
TILTING OF THE LOAD HEAD

The movement of the ends of top of the sample is shown in FIGURE V.7. As can be seen from this figure, the leading portion of the sample surface dilates while the trailing portion contracts.



NOTE

- ▲ SAMPLE F-5 $\sigma_H \approx 0.1$ P.S.I., DIAL GAUGES PLACED ON TOP PLATTEN
 ○ SAMPLE F-4 $\sigma_H \approx 6.03$ P.S.I., DIAL GAUGES PLACED ON LOAD HEAD

FIGURE V.7 - VERTICAL MOVEMENTS OF THE TOP OF THE CALCITE SAMPLE DURING DIRECT SHEAR

To further investigate this phenomenon three direct shear tests were carried out with the load head and top platten removed. The depth to the surface of the sample was measured from the top of the shear box before and after shearing. Results are given in TABLE V.5 and FIGURE V.9.

Throughout the testing programme it was observed that the top half of the shear box would rise and tilt with increasing displacement. This is illustrated in FIGURE V.8. The distance the top half of the shear box rose from its initial planar position was measured to ± 0.01 inch at the two points shown in FIGURE V.8.

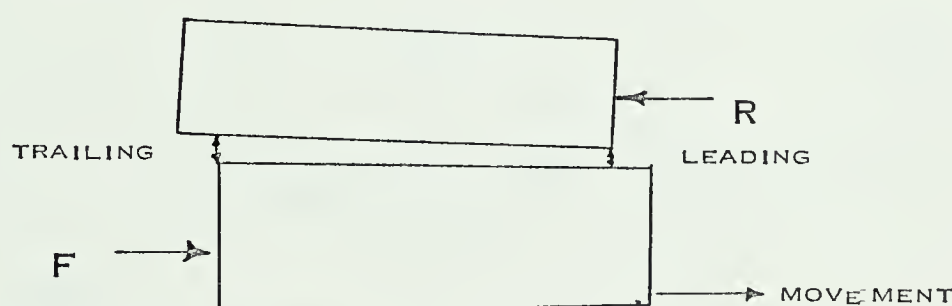


FIGURE V.8 SKETCH OF MEASUREMENTS TAKEN ON TILTING OF THE TOP HALF OF THE SHEAR BOX

No loss of sample through this tilting was observed although the amount of separation was often quite large at the trailing edge of the bottom half of the box as shown in TABLE V.6 which presents the observed data on this phenomena.

Although the results are quite erratic, it can be seen that there is a tendency for greater tilting of the box for a dense sample than for a loose sample. A sample composed of large grains tended to tilt more than a sample composed of small grains.

TABLE V.5

DIRECT SHEAR TESTS WITH NO NORMAL LOAD

TEST	HORIZONTAL DISPLACEMENT (INCH)	INITIAL VOID RATIO	AVERAGE INITIAL DEPTH (INCH)	FINAL DEPTH *	
				LEADING	TRAILING
1	0.440	0.86	0.37	0.30	0.60
2	0.440	0.81	0.40	0.21	0.61
3	0.440		0.56	0.35	0.70

NOTE: Weight of samples in all tests = 203.8 grams
Grading in samples in all tests = 50% #4 - #10; 50% #10 - #20
Tilt of the top box was negligible in all of these tests
* after horizontal displacement given in second column.

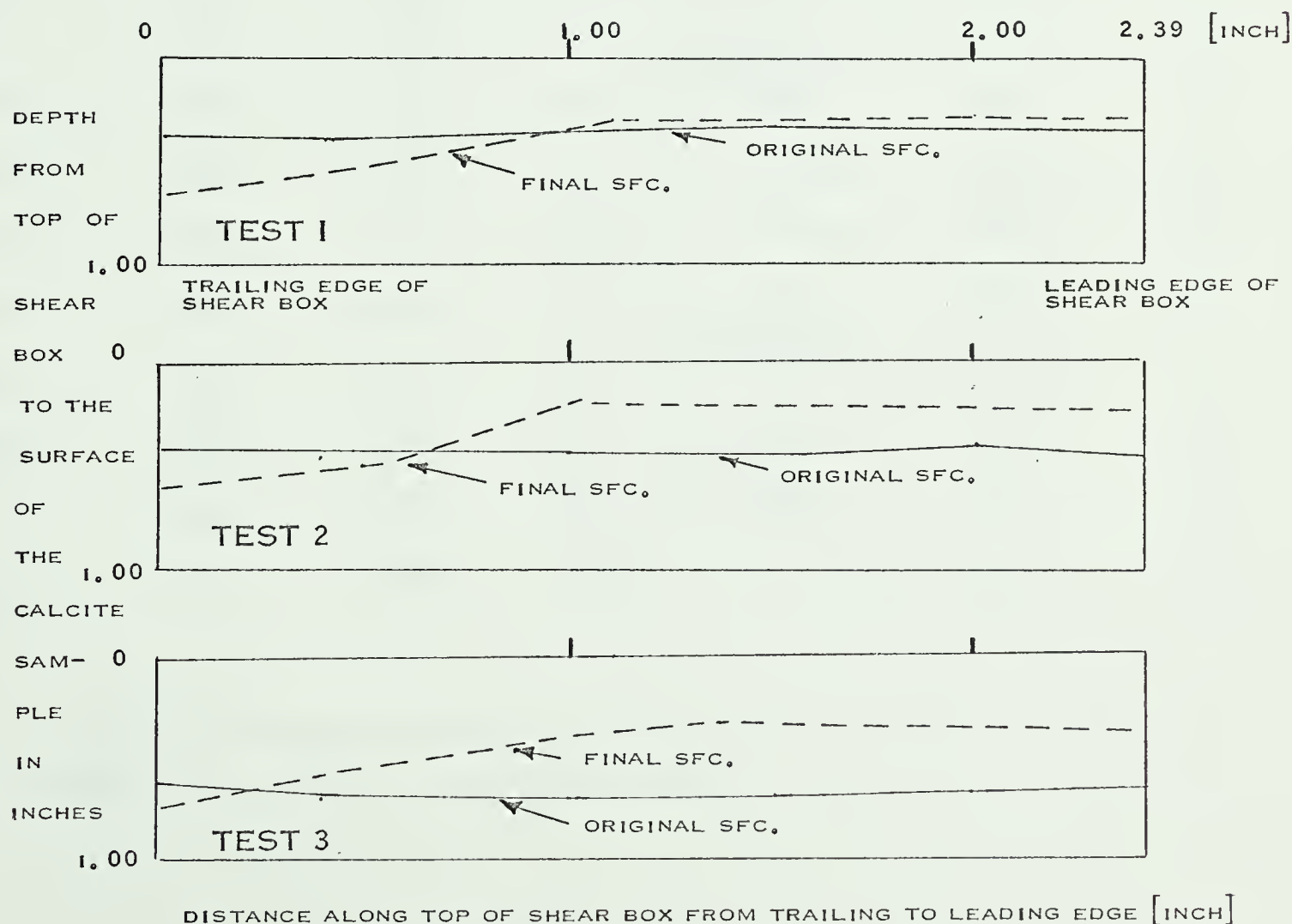


FIGURE V.9 SURFACE OF CALCITE IN DIRECT SHEAR AT
ZERO NORMAL LOAD

TABLE V.6

TILTING OF THE TOP OF THE SHEAR BOX IN TESTS UPON CALCITE SANDS

TEST	HORIZONTAL DISPLACEMENT (INCH)	INITIAL VOID RATIO	NORMAL PRESSURE (P.S.I.)	SAMPLE GRADING	RISE OF TOP LEADING (INCH)	HALF OF BOX TRAILING (INCH)
A-11	0.400	1.07	6.03	#10 - #20	0.03	0.08
A-12	0.096	0.98	6.03	#10 - #20	0.03	0.08
A-13	0.200	0.86	6.03	#10 - #20	0.01	0.07
B-1	0.280	0.65	6.03	#10 - #20	--	0.25
B-2	0.400	0.80	6.03	#10 - #20	0.07	0.12
B-3	0.200	0.67	6.03	#10 - #20	0.06	0.15
B-4	0.120	0.78	6.03	#10 - #20	0.05	0.12
C-1	0.400	1.00	6.03	#4 - #10 (angular)	0.08	0.18
C-2	0.400	0.95	6.03	#4 - #40	0.06	0.15
C-4	0.400	0.98	6.03	#4 - #40G.G.	0.03	0.08
C-5	0.400	1.05	6.03	#4 - #40G.G.	0.03	0.10
C-6	0.400	0.96	6.03	#4 - #20 (circular)	0.03	0.10
D-1	0.400	0.91	10.5	#4 - #40	0.06	0.12
D-2	0.400	0.86	17.1	#4 - #40	0.02	0.04
D-3	0.400	0.84	23.8	#4 - #20	0.03	0.08
E-2	0.400	0.90	6.03		0.05	0.16

NOTE: G. G. refers to gap-graded sample

5.7 SUMMARY:

The data presented in this chapter indicates that the process of shear failure of a granular soil in the direct shear box is a matter of some complexity. Particle rotation during shear leads to a preferred orientation of elongate particles parallel to the direction of movement. The differential movement of the surface of the sample and the tilting of the top half of the shear box indicate radically differing stress distribution throughout the sample.

One of the mineralogical characteristics of calcite is its propensity for cleavage. In shear testing it was thought that the occurrence of such phenomena might give rise to some error in the results. Examination of sections did not reveal any strong tendency for particle cleavage hence the contribution of this possibility was considered to be insignificant.

CHAPTER VI

DISCUSSION OF RESULTS

6.1 PARTICLE ORIENTATION DURING SHEAR

A study of the actual measured grain orientations as well as the photographs in APPENDIX C clearly indicates that a preferred orientation forms, with increasing horizontal displacement, in the direction of displacement. The formation of this preferred orientation can only be due to particle rotation.

A study of the polar diagrams, FIGURE V.3, shows some preferred horizontal orientation of the grains at zero displacement. This is considered to be due to the normal load applied to the sample. As the samples were sheared, the preferred direction of particle orientation can be seen to rotate until a preferred orientation parallel to the direction of shear is reached. However, even when the residual portion of the stress-displacement curve had been reached, perfect particle orientation towards the direction of shear had not been attained.

Thus as a sample of granular material is sheared in the direct shear test, particle re-orientation occurs by particle rotation and the sample develops a marked degree of anisotropy. This agrees with the findings of J. F. Arthur

at Cambridge University as reported by Roscoe and Schofield (1963).

A comparison of the behaviour of Series A (loose state) and Series B (dense state) is interesting. In order to present the state of particle orientation in a simple, but meaningful form, the following method of analysis has been adopted. The number of particles oriented at low (0° to 30°) angles to the direction of displacement has been divided by the number of particles oriented at high (60° to 90°) angles. High values of this ratio thus indicate a high degree of orientation parallel to the direction of displacement. A ratio of unity would indicate a random orientation.

TABLE VI.1

RATIO OF PARTICLE ORIENTATION FOR SERIES A AND B

SECTION	DISPLACEMENT (INCH)	(+) RATIO ¹ LOW/HIGH	(-) RATIO ² LOW/HIGH	AVERAGE
A-8	0.000	0.7	1.0	0.85
A-7	0.120	0.7	1.0	0.85
A-5	0.280	1.2	1.2	1.20
A-11	0.400	1.7	1.6	1.65
B-3	0.200	1.0	1.8	1.40
B-1	0.280	1.6	1.7	1.65
B-2	0.400	2.0	1.3	1.65
<hr/> <p>1. Refers to clockwise from the direction of displacement</p> <p>2. Refers to counter-clockwise from the direction of displacement</p>				

The results are given in TABLE VI.1. As can be seen from this table a definite increase in the degree of orientation of elongate grains, towards the direction of shear, occurs with increasing horizontal deformation. In Test Series A an average orientation ratio of 1.65 was reached at the residual.

The density of the sample appeared to affect the method of failure. Samples of loose to medium density (Test Series A, C, D and E) showed no clearly defined shear plane or shear zone. A study of the photographs in APPENDIX C shows a gradual increase in preferred horizontal orientation over most of the sample. Test Series B, in a dense state, appear to exhibit a horizontal failure plane along the plane of separation of the shear box. This failure plane appears to be a thin zone of oriented particles. Outside this zone the degree of particle orientation seems somewhat poorer than the samples of Test Series A of a comparable horizontal displacement.

The experimental results are found to confirm a portion of the theory of Bhattacharyya (1966). This theory, as presented in Chapter III, predicts the development of a virtually perfect orientation paralleling the direction of shear. A preferred orientation does in fact exist at large displacements but not every particle is in perfect alignment with the direction of shear.

Better agreement is achieved with the theory

developed by Gay (1966, 1968a, 1968b). Gay predicts that in pure shear, particles will rotate towards the direction of elongation of the sample. However, in simple shear the particles are predicted to rotate towards the direction of shear. It is important to note that only a preferred orientation is predicted, particles are predicted to rotate into and out of the plane parallel to the direction of shear. This phenomena accounts for the results observed in the polar diagrams of FIGURE IV.3 and in the photographs in APPENDIX C.

Neither the theory of Bhattacharyya or Gay takes into account the dilation of the sample which is occurring over a large portion of the sample. However, under pure shear, the particles are predicted by Gay (1966, 1968a) to rotate towards the direction of sample elongation. Therefore when the sample, or portions thereof, is dilating, particle rotation can be expected to occur toward the direction of dilation. This would account for the large number of particles oriented at large angles to the direction of shear observed at low displacements in FIGURE IV.3. It would seem likely that even at the maximum horizontal displacement reached (0.400 inch) the continuing dilation of the sample would affect the orientation of the particles. If the sample could be sheared to such a displacement that dilation had ceased, a higher degree of particle orientation could be expected.

6.2 THE EFFECT OF VARIABLES UPON THE DEGREE OF PARTICLE ORIENTATION

Test Series A established that a preferred horizontal

orientation developed with increasing displacement in a granular sample in the direct shear box. The preferred orientation can only occur by particle rotation towards the direction of horizontal deformation. It was considered advisable to study the effect of several variables upon the development of a preferred horizontal orientation during the direct shear test. These variables were sample density, sample grading, normal load and the shape of the particles. All tests were carried out at the same rate of deformation.

The effect of sample density was studied in Test Series B. The results of this series were somewhat difficult to assess as a rather small grain size (#10 - #20 sieve size) combined with insufficient grinding produced a section which was difficult to analyze. However, the three sections analyzed in TABLE V.1 (B-1, B-2 and B-3) show a similar trend to that of Test Series A; increasing preferred horizontal orientation with increased displacement. The results are not as striking as in Test Series A, it would appear that particle rotation is inhibited by increasing the density of a sample.

The effect of sample gradation and particle angularity was studied in Test Series C. A poor grading containing a large percentage of particles of too small a grain size (#20 - #40) and use of a translucent calcite made distinction of the grain boundaries difficult in most of the samples of this series. Only three sections were analyzed from this series thus few results can be drawn from so small

a grouping.

It is of interest to note that section C-1-1 (page C-17) was taken from the center of sample C-1. This section shows a poorer degree of preferred orientation than section C-1-3 which was taken 0.7 inches out towards the side of the sample. This would indicate that stress conditions vary laterally as well as longitudinally within the sample. It would be, however, unwise to draw any conclusions on the basis of just two sections.

A visual examination of the photographs of Test Series C (APPENDIX C) yields the following points:

1. Large particles surrounded by smaller particles seem to have a better degree of orientation than particles in a uniformly graded system.
2. Very elongated particles have a better examination than more spherical particles.

The effect of normal load was studied in Test Series D. The ratio of particles at low and high angles to the direction of deformation is given in TABLE VI.2. From this table there is a marked improvement in the degree of preferred horizontal orientation with increasing normal pressure for a given horizontal displacement.

Test Series E was carried out to further investigate the differences in orientation which occurred due to large amounts of horizontal displacement. An interesting feature of

TABLE VI.2

RATIO OF PARTICLE ORIENTATION FOR SERIES D AND E

SECTION	DISPLACEMENT (INCH)	NORMAL PRESSURE (P.S.I.)	RATIO		
			LOW/HIGH (+)	LOW/HIGH (-)	AVERAGE
D-1-1	0.400	6.03	1.5	2.8	2.1
2-1	0.400	17.1	1.9	2.7	2.3
3-1	0.400	23.8	2.3	6.0	4.1
E-1-1	0.400	6.03	2.0	1.0	1.5
1-2	0.400	6.03	2.6	0.8	1.7
2-2	0.400	6.03	1.5	2.5	2.0
3-1	0	6.03	2.0	1.6	1.8
5-1	0	10.5	1.2	1.3	1.2

this series is that the sections of samples E-1 and E-2 show average orientation ratios of 1.5 and 1.7 respectively after horizontal displacements of 0.400 inch. Sample E-3 (at zero displacement) shows an extremely high average orientation ratio of 1.8. It is thought that this high ratio is not significant as the two other sections at zero displacement (A-8 and E-5) yielded average orientation ratios of 0.85 and 1.2 respectively. It would appear, however, that minor details in the placing of the sample in the shear box and the subsequent handling of the shear box, before carbowaxing, could greatly affect the resulting orientations of the particles.

In summary it would appear that an increase in the density of a granular mass inhibits particle rotation during shear. An increase in the normal pressure leads to an

increased degree of particle orientation at high deformation. The effect of sample grading and angularity upon particle orientation is not clear but oblate particles appear to achieve a better degree of orientation than spheroidal particles.

6.3 THE DIRECT SHEAR TEST

The direct shear test has a number of disadvantages, chief of which is the uneven distribution of stress in the sample during shear. The results of this work show that stress conditions vary widely in the direct shear test.

In the early stages of horizontal displacement, failure does not occur along the plane of separation between the two halves of the shear box but rather along two arcs as illustrated in FIGURE VI.1. This phenomena can be seen in sections A-1, A-7 and A-9 (page C-1, C-4 and C-6) as well as in the samples of naturally occurring clay shale (APPENDIX A). The S_2 formation noted by Morgenstern and Tchalenko (1967) for the samples of Kaolin appears to be the same type of failure.

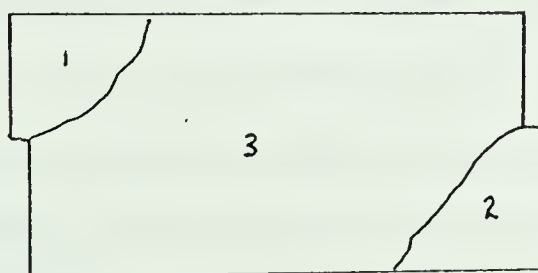


FIGURE VI.1 - SKETCH OF FAILURE ZONES AT LOW DISPLACEMENT

The reason for the formation of these failures is the basis of some controversy. Roscoe (1953) showed

that for an ideal plastic material, tension zones will exist in the upper leading and lower trailing edges of the sample at low normal loads. Therefore the failures illustrated in FIGURE VI.1 could be termed tension failures. Morgenstern and Tchalenko (1967) attribute the formation of S_2 cracks to stress concentrations.

To clarify this subject further, the behaviour of a system of perfect spheres in rhombohedral packing was considered previously in Chapter III. Consideration of the behaviour of this ideal granular material showed that zones would exist, corresponding roughly to zones 1 and 2 shown in FIGURE VI.1 which would have no intergranular forces acting in them due to the shearing force applied to the shear box. The term "dead zones" seems a somewhat better term than "tension zones" as granular soils have no tensile strength and tension forces cannot be transmitted in these zones.

From the behaviour of real soils (both granular and cohesive) as well as the predicted theoretical behaviour of an idealized granular soil, it would appear in the initial stages shear, that two zones exist in the direct shear box in which no intergranular forces act due to the shearing force causing horizontal displacement of the bottom half of the shear box. Failure occurs along the edges of these zones before the sample is failed along the plane of separation of the two halves of the shear box.

Direct shear tests were carried out at normal pressures of 0 and 6.03 p.s.i. to observe the movement

of the top surface of the sample. The results, as reported in Section IV.4 show that no dilation occurs in the trailing portion of the sample surface. This corresponds to Zone 1 of FIGURE VI.1 where the theory developed in Chapter III predicts that no dilation can occur.

Thus two independent facets in the behaviour of samples in the direct shear test (the initial arc-shaped failures and the surface movement of the sample) show clearly zones exist in the direct shear test where no intergranular forces exist due to the applied shear force or its reaction. In granular materials these are not true tension zones, although the failure which occurs in the lower leading edge of the specimen is a true tension failure. As the actual stress conditions within the sample undergoing direct shear are a poor representation of the stresses along the failure plane in an actual soil movement; the direct shear test can be said to be unsatisfactory means of measuring the peak shear strength of a soil.

6.4 DISCUSSION OF STRENGTH THEORY AND TESTING

In the calculation of the stress-displacement curve, and hence the strength parameters c and ϕ , from a direct shear test, there is an implicit assumption that the sample fails on a plane imposed by the separation of the halves of the shear box. This assumption has been shown to be incorrect, in the early stages of the test, for loose granular samples and clay shale samples (APPENDIX A). The

sections resulting from Test Series B (dense granular material) were few in number and do not show this phenomena but there is no reason to suppose that initial failure in dense granular materials does not follow the same pattern as is shown in FIGURE VI.1. If the sample fails initially along two arc-shaped failure surfaces, the first part of the stress-displacement curve, up to and including the peak strength, should be viewed with a certain amount of skepticism because the actual area over which failure is occurring is unknown. Hence the stress obtained by dividing the total shear force by the cross-sectional area is an apparent stress. Therefore, the peak strength obtained from the direct shear test is an apparent strength and is not a true measure of the shearing resistance of the soil along a planar surface.

The use of peak strengths, as determined from the direct shear test, seems unjustified when the basic assumptions of the Mohr-Coulomb failure criteria are considered. Sample dilation (as measured by one dial gauge on the load head) is a maximum at or near the peak of the stress-displacement curve. The rate of dilation decreases as the residual stress is reached. The work of Rowe (1962) and the observations noted in this thesis indicate that failure planes do not appear until the peak stress has been passed. Therefore, when the residual strength of the sample has been reached, after large horizontal displacement, at least two of the basic assumptions of the Mohr-Coulomb failure criterion appear to be valid. This point is outlined in TABLE VI.3.

TABLE VI.3

AGREEMENT OF OBSERVED SAMPLE BEHAVIOUR WITH THE BASIC ASSUMPTIONS OF THE MOHR-COULOMB

FAILURE CRITERION

ASSUMPTION	AGREEMENT AT PEAK (see TABLE II.1)	AGREEMENT AT RESIDUAL
1. No volume change during shear	Poor, rate of dilation at or near a maximum value	Good, rate of dilation approaches zero (Cornforth, 1964)
2. Sand can only slide along rupture lines	Poor, no movement apparent along expected rupture lines	Good, planar rupture line formed in cohesive soils and dense granular media (Cornforth, 1964)
3. Line of rupture independent of means obtained	Poor, considerable variation	Good for sand in triaxial and plane strain (Cornforth, 1964)
4. Line of rupture independent of intermediate principal stress	Poor, considerable variation	No data available
5. The orientation of the rupture planes can be predicted	Poor, considerable scatter	No data available

Cornforth (1964) observed for tests upon a well graded sand that plane strain tests yield peak values of the angle of shearing resistance which are more than four degrees greater than the results obtained from triaxial tests upon similar dense samples. The average difference in the residual angle of shearing resistance was only 0.7° with the triaxial test giving the higher value. Cornforth also noted failure planes did not form in the plane strain test until after the peak strength.

It would therefore appear, in light of the evidence presented in TABLE VI.3, that the use of residual angles of shearing resistance in the Mohr-Coulomb failure criterion is better justified than the use of peak angles of shearing resistance because there is better agreement with the basic assumptions of the failure criterion at residual than at peak. It is difficult to accept the Mohr-Coulomb failure criterion as being valid when peak angles of shearing resistance are used in the expression $\tau_f' = c' + \sigma \tan \phi$. At peak, most, if not all, of the basic assumptions of the failure criterion are invalid or are in poor agreement with observed data.

The evidence presented in Chapter V suggests that the direct shear test is not an entirely satisfactory method for determining the strength of a soil. The stress distribution is irregular and the "dead" or "tension" zones which exist (at low normal loads) would not exist in the field. The dilation of the sample is non-uniform with part of the sample contracting

at low normal pressures. Therefore any theoretical approach to sample behaviour is extremely difficult unless the actual conditions are grossly simplified. An illustration of this point is the fact that any analytical approach using the rate of expansion of the sample (Taylor, 1948; Bishop, 1954; Newland and Allely, 1957), as measured with one dial gauge on the load head of the shear box, is using an expression which does not represent the behaviour of the sample.

The crucial test of any means of soil testing must be whether or not it simulates the behaviour of the actual soil in the field. The direct shear test does simulate the behaviour of a real soil in a stability problem in some ways, namely:

1. A failure plane is imposed upon the soil, at least after large displacements, in a similar manner to actual field conditions (under simple shear).
2. The sample can only dilate in the vertical direction.

The direct shear test, however, does not simulate real soil behaviour in a number of ways. These are:

1. Extreme variation of stress within the sample.
2. Failure in the early stages of the test is not along the plane used in the calculation of the shear stress.
3. Tilting of the top half of the box may affect resulting strengths in a manner yet to be determined.

It can be shown that particle movement, combined with sample dilation, can explain differences in results between direct shear and triaxial tests. Nash (1953) noted that direct shear and triaxial tests on a fine sand gave much the same angle of shearing resistance. He found, however, that dense samples tested in direct shear had an angle of shearing resistance about 10% higher than that of the triaxial tests while loose samples gave results about 5% lower. The direct shear samples could only expand during shear vertically while the triaxial samples could expand radially. Thus the grains in direct shear would find it more difficult to rotate and slide during shear movement as motion in only one direction is possible. A high strength would be the result. In the triaxial test the sample is free to expand radially. Dilation would occur more easily with the particles rotating to a preferred orientation parallel with a failure plane. A lower strength can be seen to be the result. Cornforth (1964) advances much the same argument to explain why peak strengths from plane strain are higher than those obtained from triaxial tests.

6.5 DISCUSSION OF PROCEDURE

The aim of the testing procedure adopted was to produce a section by shearing the sample to a given displacement, stopping the test and then carbowaxing the sample to hold the grains in position while the sample was sliced and polished.

It is felt that only minor changes occurred in the orientation of the particles after the horizontal displacement was halted. Extreme care was taken not to jar the shear box before it was carbowaxed. Carbowax has been shown to exhibit a small amount of volume change (Quigley and Thomson, 1966). Therefore, solidification of the carbowax should have little effect upon the orientation of the calcite grains. During grinding a close watch was kept upon the relative orientation of large grains visible upon the polished face. No changes in grain position was noted in the many samples prepared.

No attempt was made to analyze the changes in void ratio and types of particle contacts with increasing displacement. After shear had ceased, the removal of the normal load and the movement of the shear box to the oven undoubtedly disturbed the sample to some degree. Particles which had reached a cubic packing would tend to return to the stable rhombohedral packing at the smallest disturbance. This tendency would be small for oblate particles, hence the particle orientation measured would not be affected. The void ratio, particle-to-particle contacts and manner of packing would be affected to some degree.

To overcome this objectionable sample disturbance and to produce truly undisturbed samples, a modification of the existing shear box is necessary. It should not prove difficult to modify the shear box so as to carbowax the sample at the end of displacement while the normal load still acts upon the sample.

The experimental technique outlined in APPENDIX B was found, generally, to yield satisfactory results once some experience was gained in the grinding of thin sections. A good deal of time could be saved, however, if a high speed lapping wheel were used. The present model revolves at about one revolution every two seconds and some forty-five minutes is required to reduce a half inch thick section to the required thickness of about 0.01 inch. High speed lapping sheels are currently used by the Research Council of Alberta and the Geology Department of the University of Alberta and would reduce the time required for grinding to about five minutes.

The table saw proved satisfactory although about ten minutes was required for each sample to be sliced into four sections. Slices could only be cut to a thickness of about 0.5 inches. Acquisition of a diamond saw operating in a varsol bath using an eight to twelve inch diameter blade would lead to the slicing of thinner sections and perhaps to some saving of time. In addition surface irregularities would be reduced and truly parallel cuts would be possible.

In the final hand grinding it was noticed there was a tendency to leave the section too thick. As can be seen in APPENDIX C a number of sections proved difficult to interpret because of excessive thickness. No evidence of this was received until after the developed photographs were returned.

Future experiments using crushed calcite, if the same method of photographing and developing is to be adopted, should use calcite no finer than a number twenty sieve size (0.0331 inch) and a large proportion of the sample should be of the number four to number ten sieve size (0.187 to 0.0787 inch).

Calcite obtained from different sources, and hence with different inclusions, has differing degrees of light transmission. Therefore, trial sections should be prepared using several combinations of calcite and cement and the variety of calcite whose grains are most clearly distinct from the cement should be used. A sample for one direct shear test weighs about 200 grams. Thus, allowing for loss of material finer than the number twenty sieve size during crushing, about two pounds of uncrushed calcite is required for each sample tested. Thus if a test programme of twenty-five samples is proposed, some fifty pounds of calcite must be obtained.

In the actual preparation of samples it is recommended that Epoxy 220 be used as a cement and that the Carbowax 6000 used be melted shortly before use.

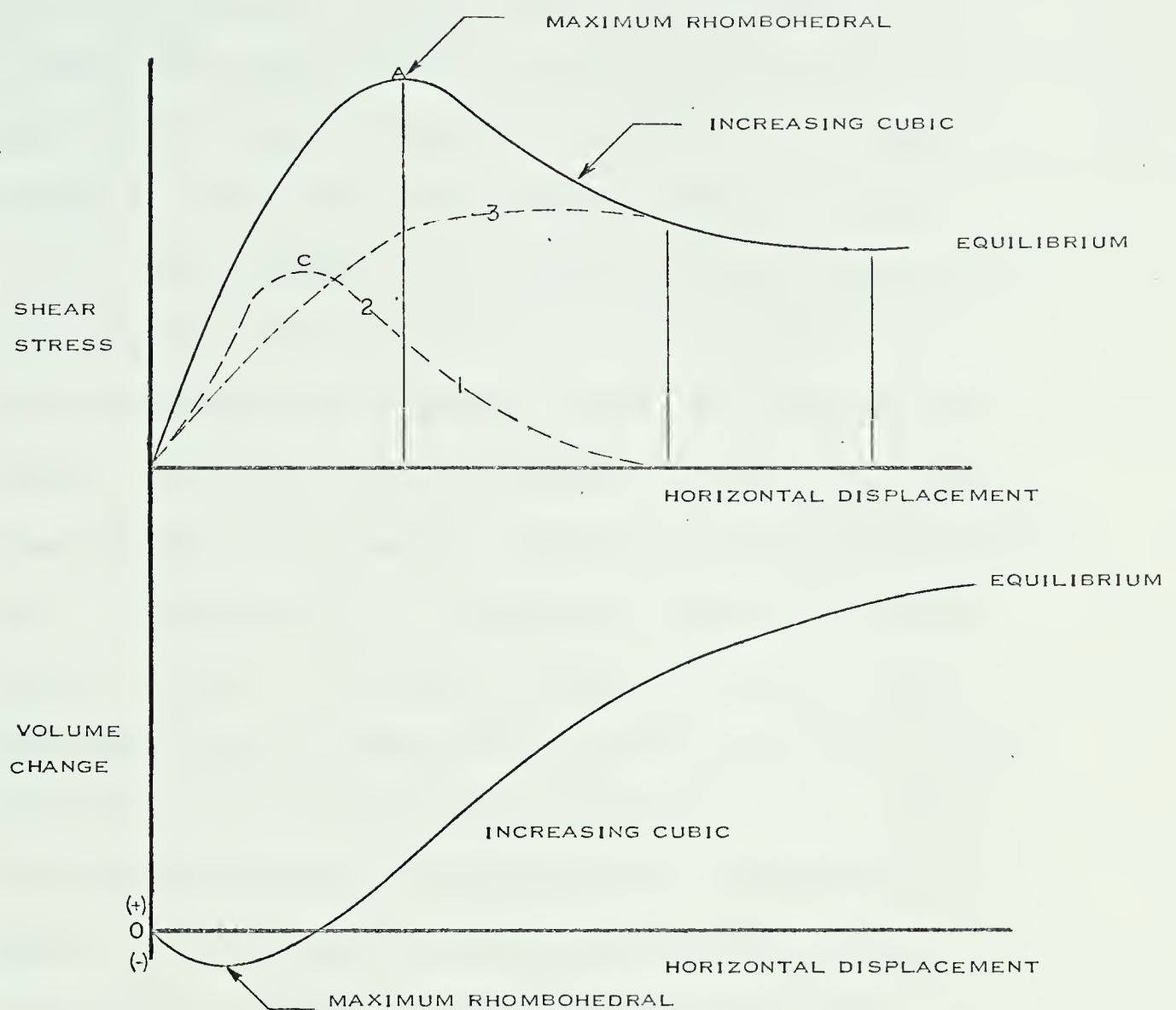
It must be emphasized that the reliability of any conclusions drawn from a study of sections is a function of the quality of the section itself. Sections in which a large number of grains are clearly distinguishable will yield concrete results, tentative results only can be obtained from indistinct sections.

6.6 CONSIDERATIONS OF THE BEHAVIOUR OF GRANULAR MEDIA IN THE DIRECT SHEAR TEST

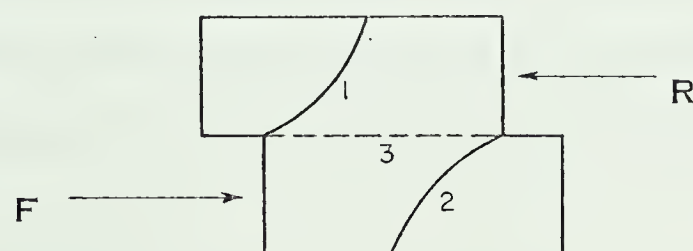
Winterkorn and Farouki (1964) show that two main packings of ideal spheres exist; rhombohedral and cubic. The rhombohedral packing is the most stable with the lowest void ratio and the highest shearing resistance. When a horizontal shearing force is applied to a rhombohedral packing, as was shown in FIGURE III.5, there will be a tendency for dilation over part of the sample surface.

A typical stress-displacement plot for a dense granular soil is shown in FIGURE VI.2(a). The packing within this sample is likely to be rhombohedral or haphazard. As the rhombohedral packing has a greater shear strength than the cubic it would appear, up to peak strength (Point A in FIGURE VI.2(a)), that the packing along the plane of failure is becoming increasingly rhombohedral. The decrease in stress past the peak would be accounted for by an increasing number of particles entering the cubic packing. Ultimately an equilibrium state should be reached at a constant, or residual level of stress, between the number of grains in the rhombohedral and the number of grains in the cubic packing along the plane of failure.

When the results of the volume change/horizontal displacement plot are viewed in the same manner, a distinct anomaly is visible by comparing the two curves at any given horizontal displacement. Sample dilation should be accompanied



[A] TYPICAL DIRECT SHEAR RESULTS



[B] SKETCH OF FAILURE SURFACES IN SAMPLE AT LOW DISPLACEMENTS

FIGURE VI. 2 - TYPICAL DIRECT SHEAR TEST RESULTS ON DENSE GRANULAR SOIL

by decreasing shear stress as the system is moving increasingly into a cubic packing. However this is not the case as can be seen in the actual plots of the direct shear results for Test Series A, B and C contained in APPENDIX D. The sample is shown to dilate long before the peak stress is reached.

This phenomena may be explained by the observations previously discussed in Section VI.3. The calculation of the stress-displacement curve assumes a flat, horizontal failure surface. Before the residual strength is reached, the sample is not failing in this manner. Therefore the stress-displacement plot is an apparent plot in the portion before the residual stress is reached. In a similar manner the volume change/displacement plot is measured by one dial gauge placed on the load head. It has been previously shown that only a portion of the sample dilates, the movement of the load head is an apparent motion and does not truly represent the behaviour of the sample. Therefore, the approaches (Lambe, 1948; Bishop, 1954; Newland and Allely, 1957) which attempt to explain the difference between "peak" and "residual" strengths, on the basis of the work done by the sample during dilation, do not take into account the actual volume-change behaviour of the sample.

The following theory is presented to explain the stress-displacement curve as obtained from the direct shear test. It is clear that initial failure, in granular material, occurs along the two arcs labelled one and two in FIGURE VI.2(b).

As movement occurs along these two arcs, the particles become aligned with the direction of movement and the stress required for movement decreases (Point C in FIGURE VI.2(a)).

It has been shown that for clays (Hvorslev, 1960) that failure in the direct shear sample progresses in from the ends of the sample. As displacement continues, the sample along the imposed failure plane is failed, from both ends towards the center, and another component is added to the stress-displacement plot. This is represented by the curve labelled three in the stress-displacement diagram (FIGURE VI.6(a)).

Along the imposed failure plane (denoted by 3 in FIGURE VI.6(b)) the stress ultimately reaches a peak and then drops due to particle orientation parallel to the direction of shear. Meanwhile the sample has moved partly into a cubic packing and there is less tendency for dilation. The "dead" zones in the sample now have intergranular forces acting between the particles and there is no further tendency for further movement along surfaces 1 and 2 as shown in FIGURE VI.2(b). The contribution of these two arc failures to the stress-displacement curve drops to zero and the residual strength of the sample is reached. Thus the stress-deformation curve as plotted in FIGURE VI.2(a) is the sum of curves 1 - 2 and 3.

It can be postulated therefore, that the peak in a stress-displacement curve is partly due to the two arc-shaped failures in the corners of the specimen added to

the effect of progressive failure along the center of the sample. This theory, although no quantitative data exists to confirm it, does explain some of the anomalies observed in the behaviour of granular materials during shear. This hypothesis strictly is applicable for light normal loads. However, even for heavy normal loads, the tendency for a "tension" type of failure should exist and may affect the early stages of the stress-displacement curve.

From considerations of particle orientation there must exist a "peak" strength for a soil being failed up along a planar failure surface. However, it should be considerably smaller than the peak strength as currently calculated in the direct shear test. If the theory advanced in this section is correct, two curved failure surfaces contribute a shearing stress component to the shearing stress contributed by progressive failure along the horizontal plane.

6.7 RESUME

Current design practice, as has been discussed in Chapter II, uses peak strength parameters and rather large factors of safety. It would appear to the author, that use of the residual strengths and lower factors of safety would be more rational an approach to the design of earth structures.

The results of this research and study of the literature, indicate that the residual strength measured in the direct shear test, is a true measure of the ultimate shearing resistance of a soil along a planar failure surface. When the residual strength of a soil is reached the evidence

available suggests that most, if not all, of the assumptions of the Mohr-Coulomb failure criterion are valid. If the assumptions which the criterion is based upon are valid, it follows that the criterion itself should be valid.

The Mohr-Coulomb failure criterion is a good first approximation to the peak strength of a soil when the peak strength parameters are used. However, peak strengths have been shown (Skempton, 1964) to lead to erroneous factors of safety in some stability analyses. The experimental results of this thesis, as discussed previously, appears to indicate that the peak strength, as measured in the direct shear test, is an apparent strength and is greater than the peak shearing resistance of a soil along a planar failure surface.

It would appear that "true" peak strength does exist for soils, however, the difference between this "true" peak strength and the residual strength is not presently known. Design of earth structures using residual strengths would, undoubtedly, prove ultra-conservative in many cases and result in very expensive structures. In most designs some part of the "peak" strength could be used with a low factor of safety such as 1.25 to 1.40. In designs of costly and important structures (such as earth dams) a larger factor of safety could be used or the structure could be designed using residual strength parameters and a small factor of safety (such as 1.10).

It would be desirable if further research could

ascertain the actual peak value for the shearing resistance of a soil along planar failure surface. If this value was accurately known, economical and safe design of soil structures would be optimized and the present necessary practice of over-design could be eliminated.

CHAPTER VII

CONCLUSIONS

On the basis of the experimental results of this thesis it is felt that the following conclusions are justified for the materials and test conditions studied.

1. The use of sections to study the mechanism of failure of soils, especially granular soils, appears to be capable of leading to a better understanding the basic strength properties of soils and strength testing methods.
2. Granular particles, subjected to a state of simple shear, rotate towards the direction of shear. A preferred, but not perfect, orientation of the grains towards the direction of shear results.
3. Granular soils appear to behave like a viscous fluid when subjected to simple shear. The mathematical theory of Gay (1966, 1968a, 1968b) is confirmed qualitatively.
4. The amount of particle rotation in a sheared granular

soil appears to be affected by the initial density of the sample. Particles in a dense sample seem to rotate to a lesser degree than the particles in a loose sample and a poorer degree of orientation results in a dense sample.

5. The normal load to which a sample of granular soil is subjected appears to affect the amount of particle rotation. An increase in normal load appears to result in a better degree of preferred orientation at large horizontal displacements.
6. The stress conditions vary widely in a direct shear sample during shearing. For tests where the normal pressure is small, zones exist in the sample at small displacements where no intergranular forces act due to the application of the shearing force.
7. Initial failure in the direct shear test occurs along two arc-shaped surfaces in the upper-trailing and lower-leading portions of the shear box. Failure does not initially occur along the horizontal plane of separation of the two halves of the shear box.
8. The peak strength as conventionally measured in the direct shear test, and hence the peak angle of shearing resistance, seems to be an apparent

strength which is greater than the peak strength of the sample along a planar failure surface. A true "peak" value of shear strength should exist which is somewhat higher than the residual strength but it is probably lower than the peak value of strength measured by current direct shear testing procedure.

9. The Mohr-Coulomb failure criterion may not be valid when the peak angle of shearing resistance is used.
10. The residual angle of shearing resistance appears to be a true measure of the ultimate strength of a sample along a planar failure surface. The Mohr-Coulomb failure criterion appears to be valid when the residual angle of shearing resistance is used.

CHAPTER VIII

RECOMMENDATIONS

This research has been, of necessity, a preliminary programme. While the procedures used produced satisfactory results it is felt that a good deal of improvement is possible. It is therefore recommended that:

1. Further investigation of soil structure and strength-deformation characteristics be carried out using the same general procedure. Further testing of granular materials should present few problems although sectioning of silt and clay samples may present some difficulties.
2. Further investigation into the failure of granular media in the direct shear test be conducted to augment the work carried out in this thesis.
3. Investigation into the failure of triaxial samples be conducted and the results be compared with observations from direct shear samples.
4. Modification of direct-shear machine be made to enable the sample to be carbowaxed at the

end of displacement without disturbance of the sample or removal of the normal load. Meaningful studies of void ratio changes and particle-to-particle contacts would then be possible.

5. Certain improvements in equipment be undertaken. A high-speed lapping wheel and diamond saw should be purchased. It would be desirable to train a technician to produce (thin) sections of good quality.
6. Future refinements in the study of sample failure and particle movement of granular media be considered. Design and construction of a shear box with transparent walls of plastic or glass would allow observation of actual particle motions during shear. Edge effects would affect the results to some degree but it is felt worthwhile results would be achieved.
7. Studies be conducted on particle orientation of granular media at large displacements by use of a "ring shear" apparatus.
8. Studies be undertaken to determine a means of finding the peak shearing resistance of a soil along a planar failure surface.

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APPENDIX A
SHEAR STRENGTH TESTING AND CARBOWAXING
OF SILTS AND NATURAL CLAYS

A.1 INTRODUCTION

Originally this research was intended to study void-ratio changes during shear. It was proposed to use a pure silt, unconfined compression and microscopic analysis of the resulting thin sections. As is shown in this Appendix, the use of these materials was not feasible and its use was abandoned. It was felt useful, however, to record these tests.

The soil used was a pure silt from Mile 980 of the Alaska Highway. This soil was not satisfactory as after shearing, it was found to disintegrate when placed in liquid Carbowax 6000.

Naturally occurring cohesive soils were sheared in direct shear, sectioned and studied. From the samples studied it is apparent that the concept of a single shear "plane" is incorrect. Samples from the direct shear machine exhibited irregular shear surfaces.

Natural slickensides opened when the samples were placed in liquid carbowax. This would appear to indicate a good deal of movement had occurred along these slickensides.

A.2 PROCEDURE

The Alaska Highway Silt was obtained from approximately Mile 980 of the Alaska Highway by Dr. S. Thomson. The material was ground and sieved through a number forty sieve. The natural soil exhibited little dry strength, was light grey in colour and had a silky texture. Routine classification tests were carried out in accordance with A.S.T.M. (1958) specifications.

The results are in TABLE A-1. Cylindrical samples were prepared by consolidating the soil from the liquid limit in two inch inside diameter perspex tubes. Three samples were extruded, and tested in unconfined compression.

This procedure proved unsatisfactory for reasons which will be presented in the next section.

TABLE A-1

CLASSIFICATION TESTS ON ALASKA HIGHWAY SILT

SPECIFIC GRAVITY	2.67	
ATTERBERG LIMITS		
LIQUID LIMIT	25.3%	
PLASTIC LIMIT	21.0%	
PLASTICITY INDEX	4.3%	
GRAIN SIZE DISTRIBUTION ⁽¹⁾		
% SAND SIZES	12	
% SILT SIZES	85	
% FINER THAN 2 MICRONS	5	
MINERALOGICAL COMPOSITION	30%	
QUARTZ		
PLAGIOCLASE		APPROXIMATELY
CHLORITE		EVENLY
BIOTITE		DISTRIBUTED
AMPHIBOLE		
CLAY MINERALS	NIL	
<hr/>		
1. M.I.T. Grain Size Scale		
2. From Balanko (1968) with permission		

Samples of undisturbed cohesive soils were tested in direct shear. Portions of Shelby tubes, containing undisturbed samples of Peace River Clay and Little Smokey Clay Shale, were presented to the author by Messrs. D. Pennell and D. W. Hayley. The samples were trimmed and mounted in the same direct shear box in which the calcite samples were tested. The properties of these soils are shown in TABLE A-2:

TABLE A-2

PROPERTIES OF NATURALLY OCCURRING UNDISTURBED SOILS

PHYSICAL PROPERTY	LITTLE SMOKEY CLAY SHALE ⁽¹⁾	PEACE RIVER CLAY ⁽²⁾
Natural Water Content	21	32
Liquid Limit	55	73
Plastic Limit	31	27
Plastic Index	24	46
Liquidity Index	.28	-
% Sand	5-6	4
% Silt	51-57	31
% Clay	38-43	65
Activity	0.60	0.7
Specific Gravity	2.71	2.75
Bulk Density (p.c.f.)	139	118
Sample Location	Little Smokey Bridge	Peace River Town Slide Area
(1) See Hayley (1968) (2) See Pennell (1968)		

The testing programme on these two materials is outlined in TABLE A-3 as follows:

TABLE A-3

TESTING PROGRAMME UPON UNDISTURBED SOILS

SAMPLE (1)	NORMAL PRESSURE (2)	HORIZONTAL DISPLACEMENT (3)	RATE STRAIN (4)	MACHINE (5)	REMARKS (6)	LOCATION (7)
PEACE RIVER CLAY						
1	113 p.s.i.	0.245 inch	2.375 inch/ day	Double Shear	Sketched	-
LITTLE SMOKEY CLAY SHALE						
2	6.03 p.s.i.	0.280 inch	0.008 inch/ sec.	Single Shear	Photographed	Hole LS9 Depth 85'
3	6.03 p.s.i.	0.360 inch	0.008 inch/ sec.	Single Shear	Disintegrated in carbowax	Hole LS9 Depth 85'
4	6.03 p.s.i.	0.200 inch	0.008 inch/ sec.	Single Shear	Photographed	Hole LS9 Depth 86'

After testing each sample was extruded from the shear box and placed in liquid Carbowax 6000 at 160°F. for fourteen days. On removal the samples were sketched and samples L.S.C.S. #2 and #4 were slabbed, polished and the polished faces were photographed data sheets are included at the end of this appendix.

A.3 OBSERVATIONS

(1) Alaska Highway Silt: It proved impossible to prepare sections of this soil. The samples, when extruded from the perspex consolidation tubes, seemed solid enough for testing. However, any disturbance had the effect of dilating them.

When placed in liquid carbowax, after testing, all samples disintegrated. The use of this soil was therefore abandoned.

(2) Peace River Clay: A very poor thin sample resulted from this soil. It was noted that large cracks opened in the sample, after several days in liquid carbowax, due to natural slickensides. A section was made of this soil but excessive grinding almost entirely removed the soil.

(3) Little Smokey Clay Shale: Photographs of polished faces of L.S.C.S. #2 and #4 are shown in PLATES A-1 and A-2. No thin sections were made. Sample L.S.C.S. #3 behaved in a very similar manner and is sketched in PLATE A-3. No photograph of this sample is available as it was extensively shattered and disintegrated in the carbowax.

A typical stress-displacement curve for these samples is

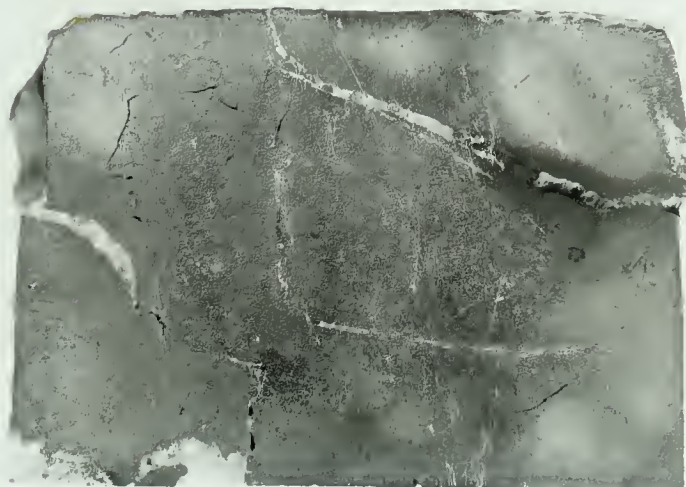


PLATE A.1 - L.S.C.S. SAMPLE NUMBER 2

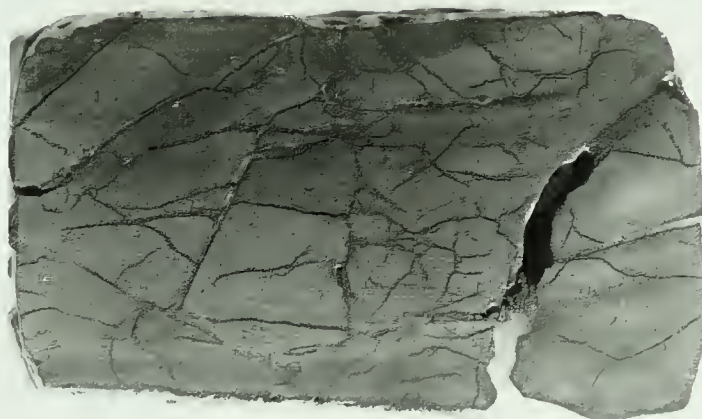


PLATE A.2 - L.S.C.S. SAMPLE NUMBER 4

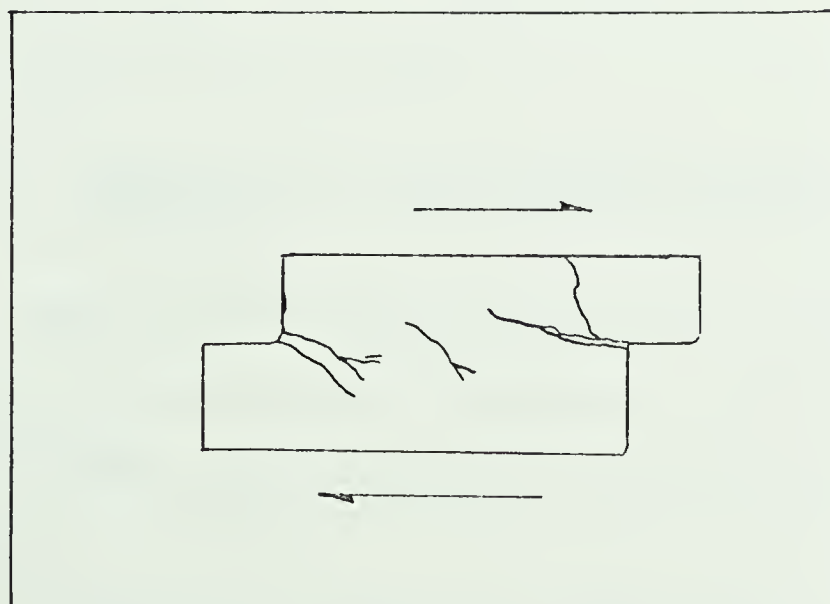


PLATE A.3 - L.S.C.S. SAMPLE NUMBER 3

shown in FIGURE A-1.

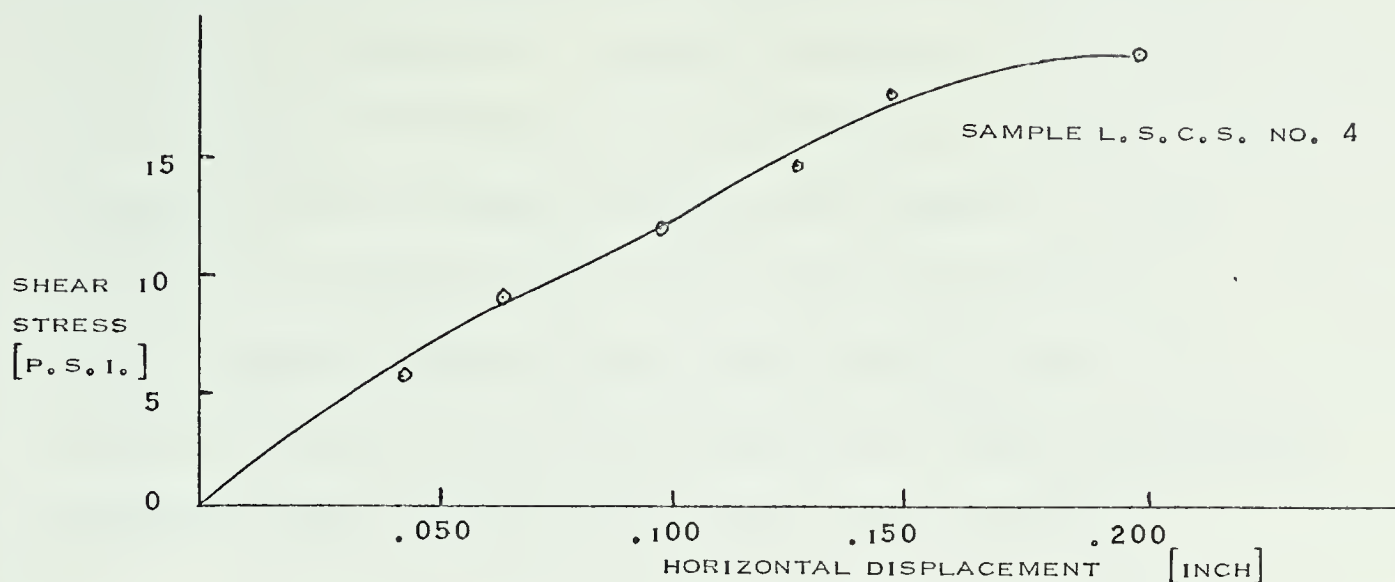


FIGURE A-1 STRESS-DISPLACEMENT CURVE FOR CLAY SHALE SAMPLE A.4 DISCUSSION

From PLATES A-1, A-2 and A-3 it is apparent that the samples have a tendency to split into three zones as shown below:

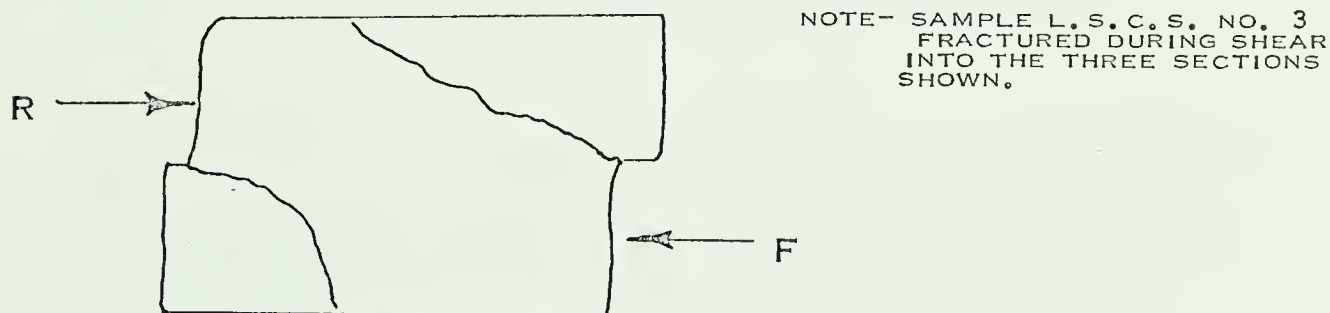


FIGURE A-2 SKETCH OF LONGITUDINAL SECTION OF L.S.C.S. SAMPLES

The fractures sketched above seem to be analogous with the S₂ formation noted by Morgenstern and Tchalenko (1967). No further interpretation is possible as no thin sections were made.

Whether these features are due to stress concentrations, as postulated by Morgenstern and Tchalenko, or

tension failures, is debatable. However, the size of these fractures would argue for a mode of failure distinct from that causing the fractures in the center of the specimen. The irregular shape of these fractures, when compared to those formed in consolidated kaolin (Morgenstern and Tchalenko, 1967), shows the effect of over-consolidation, slickensides, and local variation in the soil properties. This is especially marked in Sample L.S.C.S. #4 as can be seen in PLATE A-2.

The sample of Peace River Clay was interesting because of the large number of slickensides. Upon trimming it was noted that the sample was heavily slickensided. After several days immersion in liquid carbowax it was noted that the slickensides opened more than the shear planes in the sample. This would appear to indicate that:

- (1) the cohesive forces along the slickensides are very low.
- (2) that much more movement has occurred along the slickensides than the 0.245 inches which occurred along the shear planes of the direct shear samples.
- (3) in a heavily slickensided soil, the pattern of the slickensides may control the method of failure of the soil.

A.5 CONCLUSIONS

1. It appears impossible to use Carbowax 6000 to make thin sections of pure silt samples consolidated from the liquid limit.

2. Before the peak in the stress-displacement curve of the direct shear test, failure does not occur along the plane of separation between the two halves of the box. Failure occurs as two arc-shaped cracks which appear to be due to either stress concentrations or tension zones.

3. In over-consolidated soils slickensides may affect the mode of failure considerably. Considerable movement seems to have occurred along these slickensides.

A.6 RECOMMENDATIONS

1. Further direct shear testing be carried out on both naturally occurring and artificially sedimented cohesive soils. The effect of over-consolidation upon the mechanism of failure be noted. Thin sections should be taken from each specimen.

2. The effect of rate of displacement and normal load upon the mode of failure be studied.

3. Triaxial tests be used and the resulting thin sections be compared with those from direct shear tests.

4. The effect of natural slickensides upon failure surfaces be studied.

APPENDIX B

DETAILS OF SHEAR TESTING AND THIN SECTIONING PROCEDURE

B.1 INTRODUCTION

The aim of the test programme was to produce longitudinal sections of the calcite with the grains at orientation they had reached when horizontal displacement ceased. It is felt this was largely achieved, however, this is fully discussed in Chapter V.

B.2 SAMPLE PREPARATION

The original samples of calcite were fractured into approximately four inch pieces by use of a hammer and chisel. These were crushed in a Mossco crusher with the jaws closed to one-eighth of an inch clearance. No trouble was encountered crushing the calcite.

The resulting material was sieved through a series of U.S. standard sieves. The crushed calcite was separated and stored in polyethylene bags in the following sizes: number four to number ten, number ten to number twenty, number twenty to number forty. Grain sizes smaller than the number forty were discarded and material larger than the number ten sieve size (1.651 millimeters) was discarded.

Samples were mixed to desired grain size curves by weight. It was found that a weight of ± 0.02 gm. could be obtained.

B.3 DIRECT SHEAR TEST

A diagram of the direct shear machine is given in FIGURE B-1. The bottom platten A was taped in position with masking tape being placed on the bottom of the platten A and the bottom of the shear box.

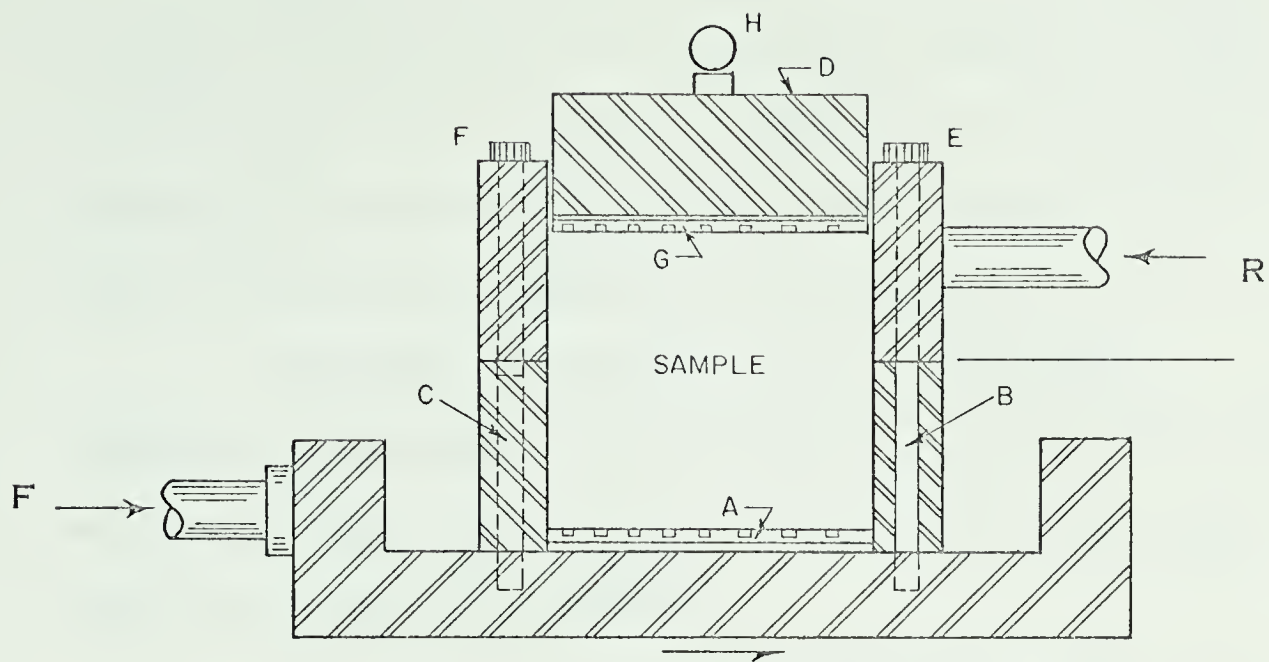


FIGURE B. 1 - SECTION OF THE DIRECT SHEAR MACHINE

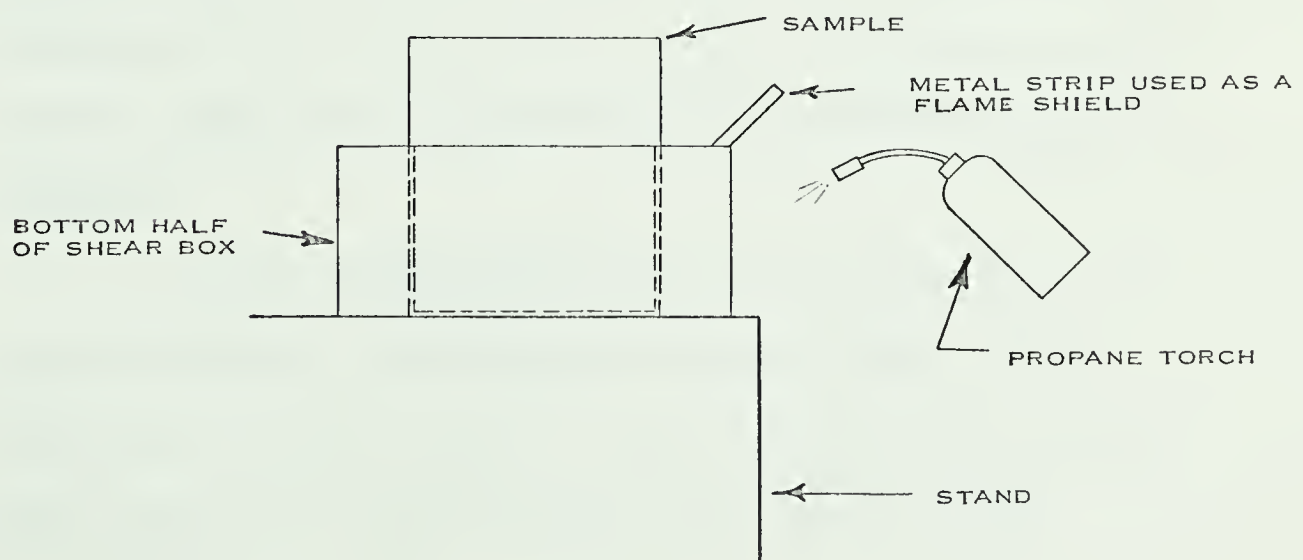


FIGURE B. 2 - METHOD OF REMOVING THE SAMPLE FROM THE DIRECT SHEAR BOX

The box was assembled with screws B and C tightened to hold the bottom half of the shear box in position. Screw C was then loosened and withdrawn. The top half of the box was placed in position and screws E and F were tightened. The height of the box was measured to ± 0.01 inch.

The sample was placed in the box after being weighed to ± 0.01 gram. In Test Series A, C, D and E the sample was poured in and then levelled. In Test Series B the sample was vibrated for thirty seconds.

The shear box was placed in position on the rails of the machine. The top platten G and the load head D were applied. The normal load was applied and the dial gauge H was placed in position and zeroed. The pins E and F were removed and the sample was ready to test.

The test was carried out rotating the handcrank at one revolution per second. This gave a rate of horizontal displacement of 0.008 inches per second. The proving ring and dial gauge were read alternately for load and vertical movement.

At the predetermined horizontal displacement the test was stopped. The shearing force was backed off and the normal load was removed. Prior to removal of the shear force, the tilt of the top half of the shear box was measured.

The shear box was removed from the machine and the load head and top platten were removed. The box was carefully placed in a pan and then into an oven at 160°F.

B.4 CARBOWAXING

Previous to the test, dishes were filled with the flaked carbowax and placed in an oven at 160°F. The carbowax required about two hours to melt.

The shear box was removed from the oven after fifteen to twenty minutes and placed on a table. Liquid carbowax was poured over the top of the calcite until the brim of the shear box was reached. This level was maintained until the carbowax solidified. The time required for solidification of the carbowax was about forty-five minutes.

B.5 SAMPLE REMOVAL FROM SHEAR BOX

The solidified carbowax and calcite formed a sample which had the hardness of talc. Heat had to be applied to the box to melt the carbowax along the outside of the sample in order to remove the sample. The procedure followed is outlined below:

1. A small propane torch was used to heat the top half of the shear box. A low flame was used and each side was heated for about fifteen seconds.

2. The upper portion of the shear box could then be lifted off the sample vertically. If the proper amount of heat was used, no slumping of the sample resulted.

3. The one screw, B, holding the bottom half of the box to the base was withdrawn. The base was heated with the propane torch and the bottom half of the box could then be removed.

4. The bottom half of the shear box, holding the sample, was placed on a stand and heated. A metal strip was used as a flame protector as shown in FIGURE B-2. The masking tape on the bottom of the platten and the shear box was removed and the bottom platten was heated. The bottom platten was then pressed upon and the sample extruded from the bottom half of the shear box.

5. After the sample cooled, the bottom platten was heated and removed from the bottom of the sample.

6. The sample was then marked with a marking pencil to show sample orientation and the direction of shear. The sample was now ready for slicing.

This procedure was found to give extruded samples which showed no sign of sample disturbance or slumping. If, however, excessive heat was used and more than the outer skin of carbowax was melted, a large amount of slumping occurred along the edges of the sample. In a few cases melting of the sample occurred.

B.6 PREPARATION OF SECTIONS FOR OPTICAL STUDY

The next stage in the programme was the cutting of samples along their longitudinal axis to produce a view of the entire area of the sample parallel to the direction of movement.

No suitable diamond saw being available, a table saw with an eight inch diameter blade was used. This worked well

provided a slow rate of feed was used. The blade was found capable of cutting through particles of calcite. Some sample disturbance undoubtedly occurred adjacent to the blade but this zone was removed before the sample was mounted.

Normally each sample was cut into four equal slices. The slices selected for mounting were treated in the following manner.

1. The face selected for mounting was ground for several minutes on a lapping wheel using number 220 silicon carbide grinding powder with varsol used as a lubricant. About 0.1 inch of thickness was removed.

2. The face selected for mounting was then hand polished on a glass plate using 220 and 600 silicon carbide. The process of Mitchell (1955) was used except that varsol was used as a lubricant instead of kerosene.

3. The face was cleaned with varsol and dried. It was glued on a 25 by 75 millimeter glass microscope slide. Three cementing agents were used. The procedures are given below:

- (a) Lakeside 70C (obtainable from Hugh Courtright and Company, 1209 W. 74th Street, Chicago, Illinois, 60636) is a thermosetting resin cement. A glass slide was heated on a hot plate to about 170°F. and the cement melted upon the slide and spread evenly. The sample was placed upon the slide and both were immediately removed from the hotplate to reduce the melting of the carbowax minimum. While the cement was still hot and liquid,

pressure was applied to the back of the glass slide to remove air bubbles.

- (b) Permunt (obtainable from Fischer Scientific Company) is a colourless cementing agent used mainly in the preparation of biological specimens. It was placed upon the glass slide and the sample was placed polished face down upon it. Care must be taken to avoid entrapping air bubbles. Ten days were found to be necessary for the curing of the Permunt.

- (c) Epoxy 220 (obtainable from Rock and Gem Supply (Edmonton) Ltd., 7605 - 104th Street, Edmonton) is an epoxy resin. Its two components were mixed in equal proportions upon the glass slide and the polished face of the sample was placed upon the slide. The cement required about six hours at room temperature to set.

4. The slice was ground to a thickness of about 0.01 of an inch, using number 220 abrasive, upon the lapping wheel and glass plate. Final grinding of the sample used number 600 abrasive. Grinding was halted when the calcite grains could be clearly differentiated when the slide was held up to the light.

5. At conclusion of the grinding the section was cleaned with varsol. No cover glass plate was used. The section was then ready for study.

This procedure proved satisfactory as the calcite particles were held in place by the carbowax during grinding. Some difficulty was encountered with the cementing agents.

Lakeside 70C required heat for its application and, while it is unlikely any large amount of sample disturbance occurred, it seemed desirable to use a non-thermsetting cement. Lakeside 70C, however, set immediately upon cooling and no delay was incurred in the procedure. The cement usually entrapped more air bubbles than either Permunt or Epoxy 220.

Permunt required at least ten days to set and, even then, proved to be a weak bonding agent. A considerable degree of plucking occurred during the final grinding stages of samples mounted with Permunt. Section A-7 is a good example of this drawback to the use of Permunt. However, a better contrast between grains was achieved by using this cement rather than Lakeside 70C.

Epoxy 220 proved very satisfactory as it has a short setting time (six to eight hours) and a good bond. No plucking occurred of samples mounted with this cement. Several days after mounting small crystals appeared between the glass slide and the sample indicating the epoxy might be reacting with the carbowax. This, however, does not seem to be a serious drawback as the sections had been photographed before this occurred. However, samples mounted with this cement might be damaged after long periods of storage.

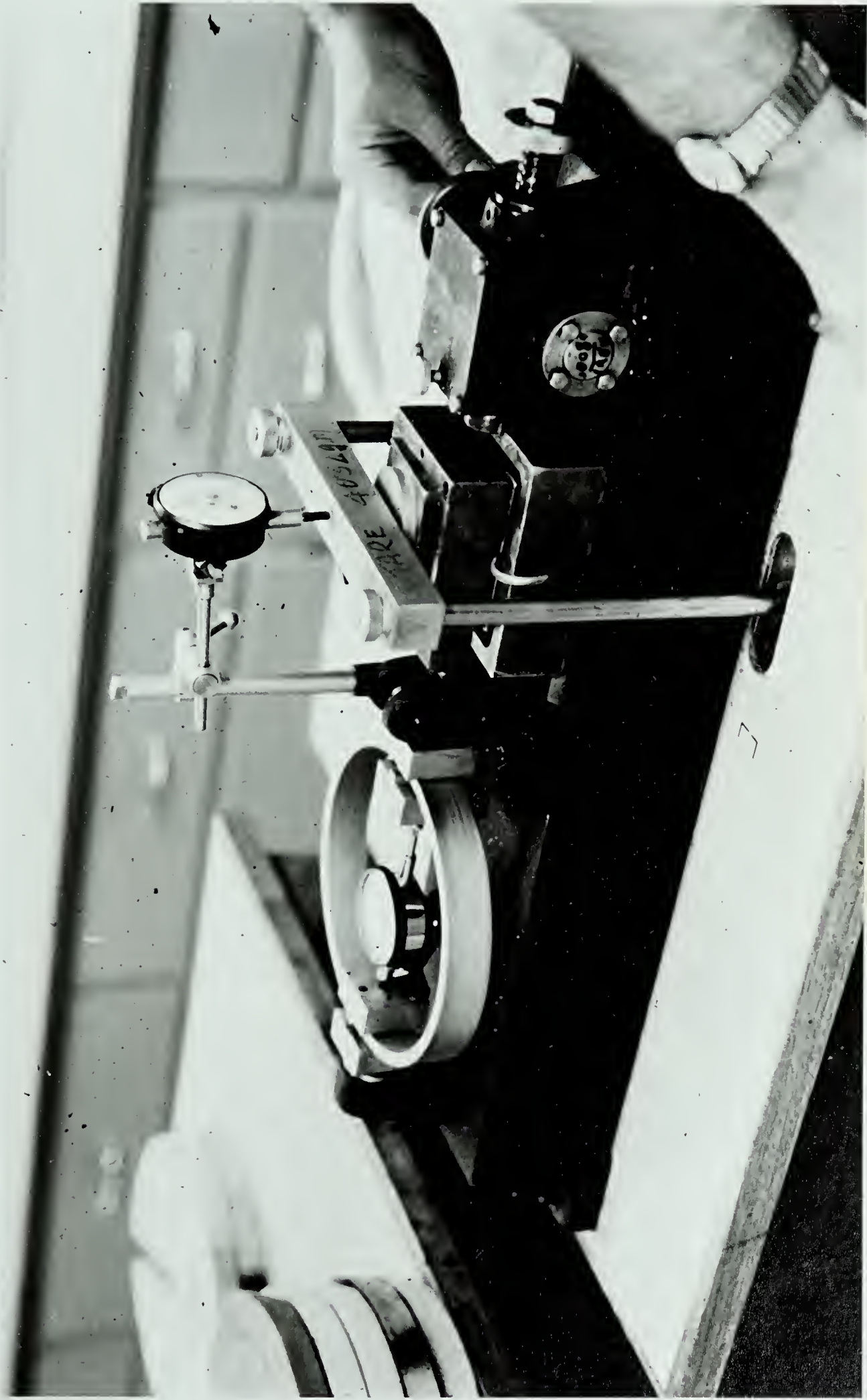
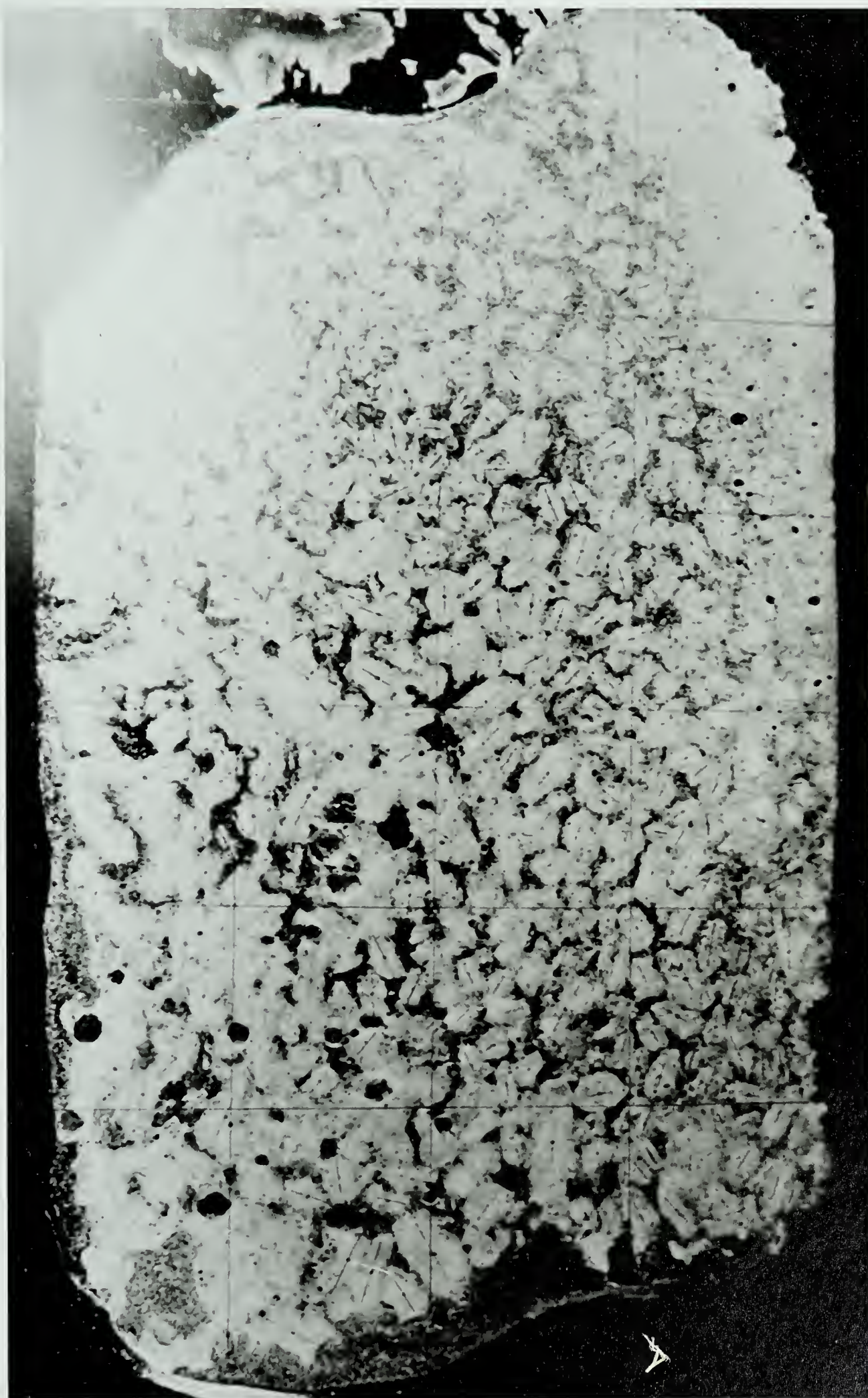


PLATE I - DIRECT SHEAR
MACHINE

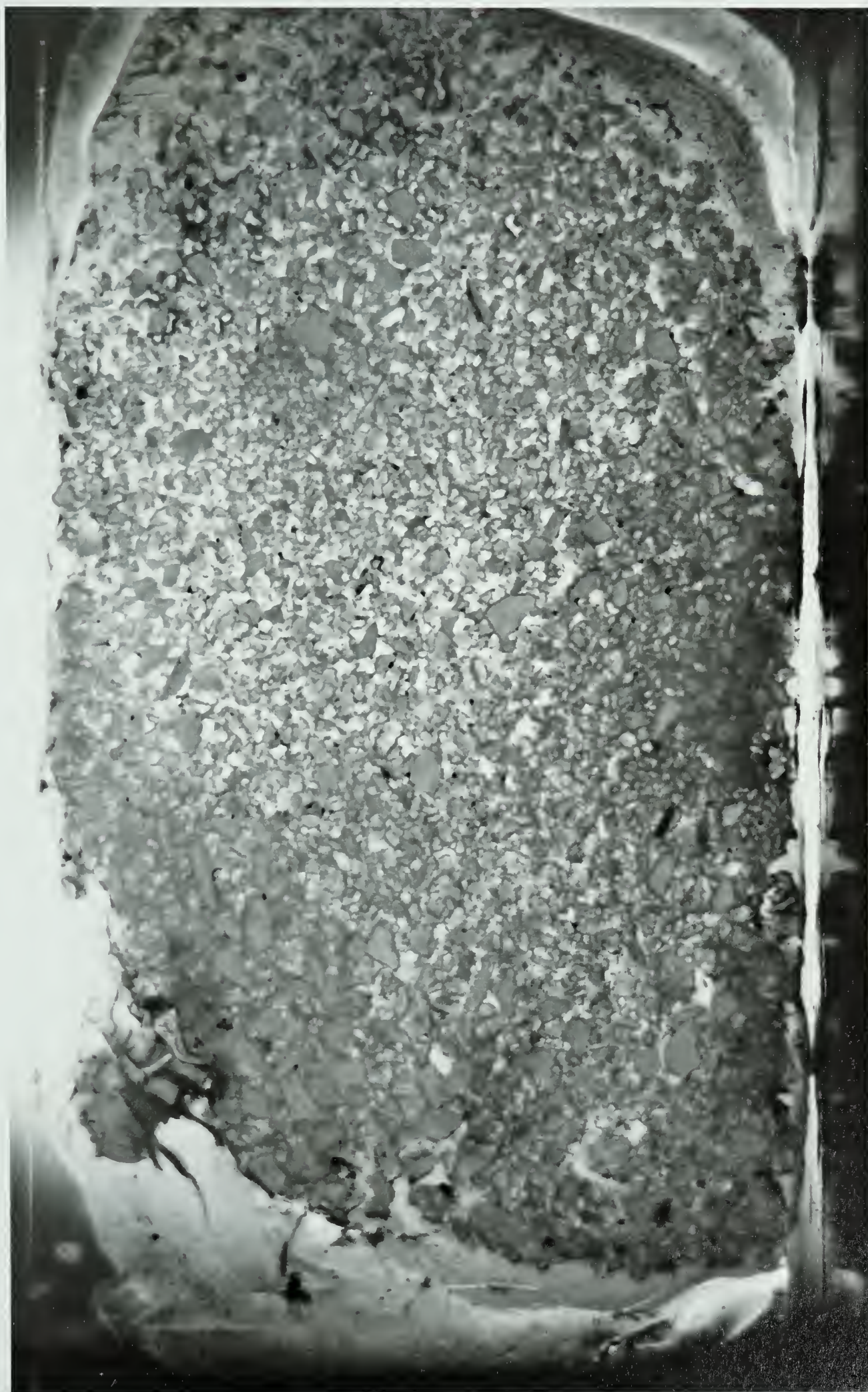


PLATE 2 - GRINDING SECTION
ON LAPPING
WHEEL

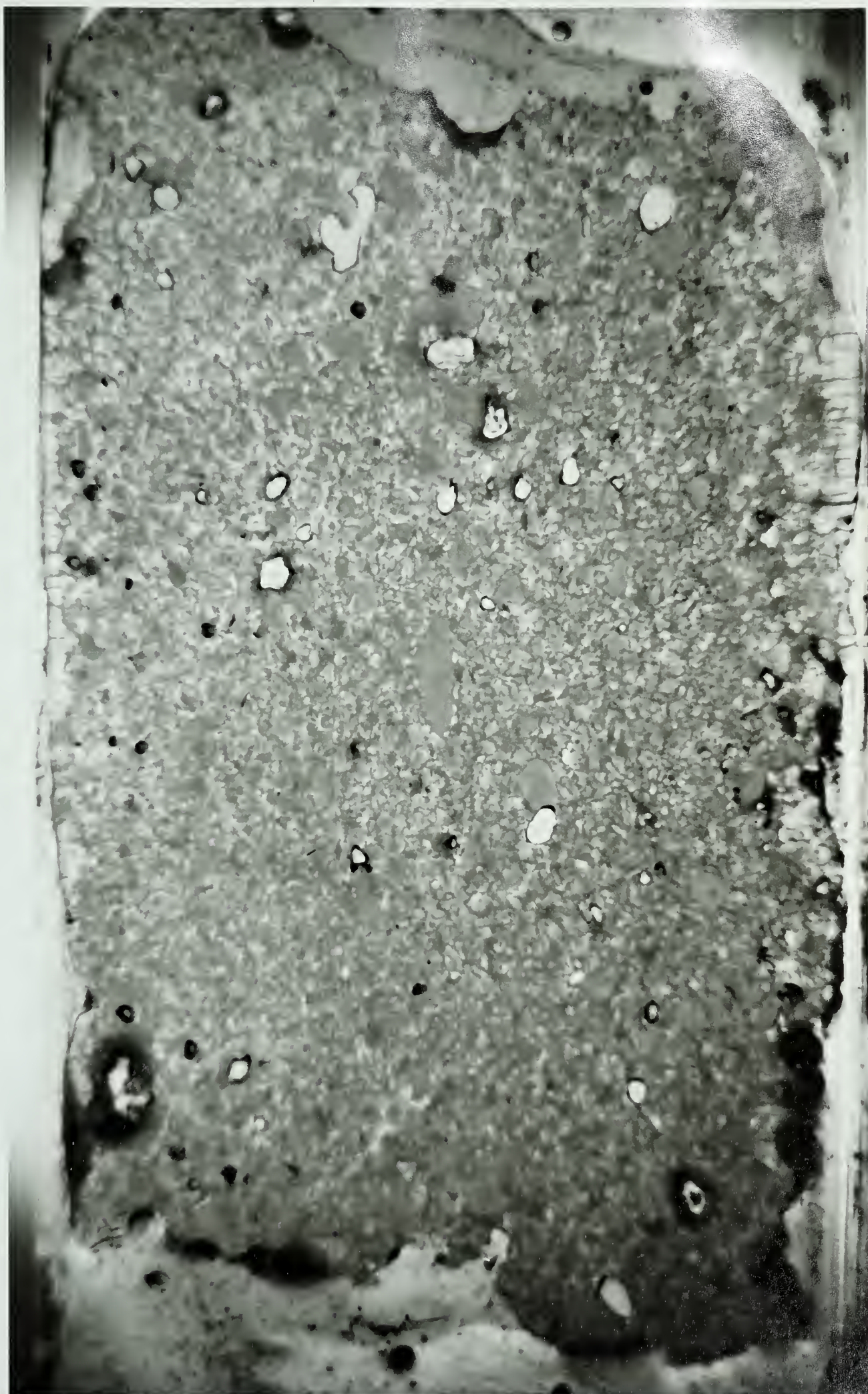
APPENDIX C
PHOTOGRAPHS OF SECTIONS



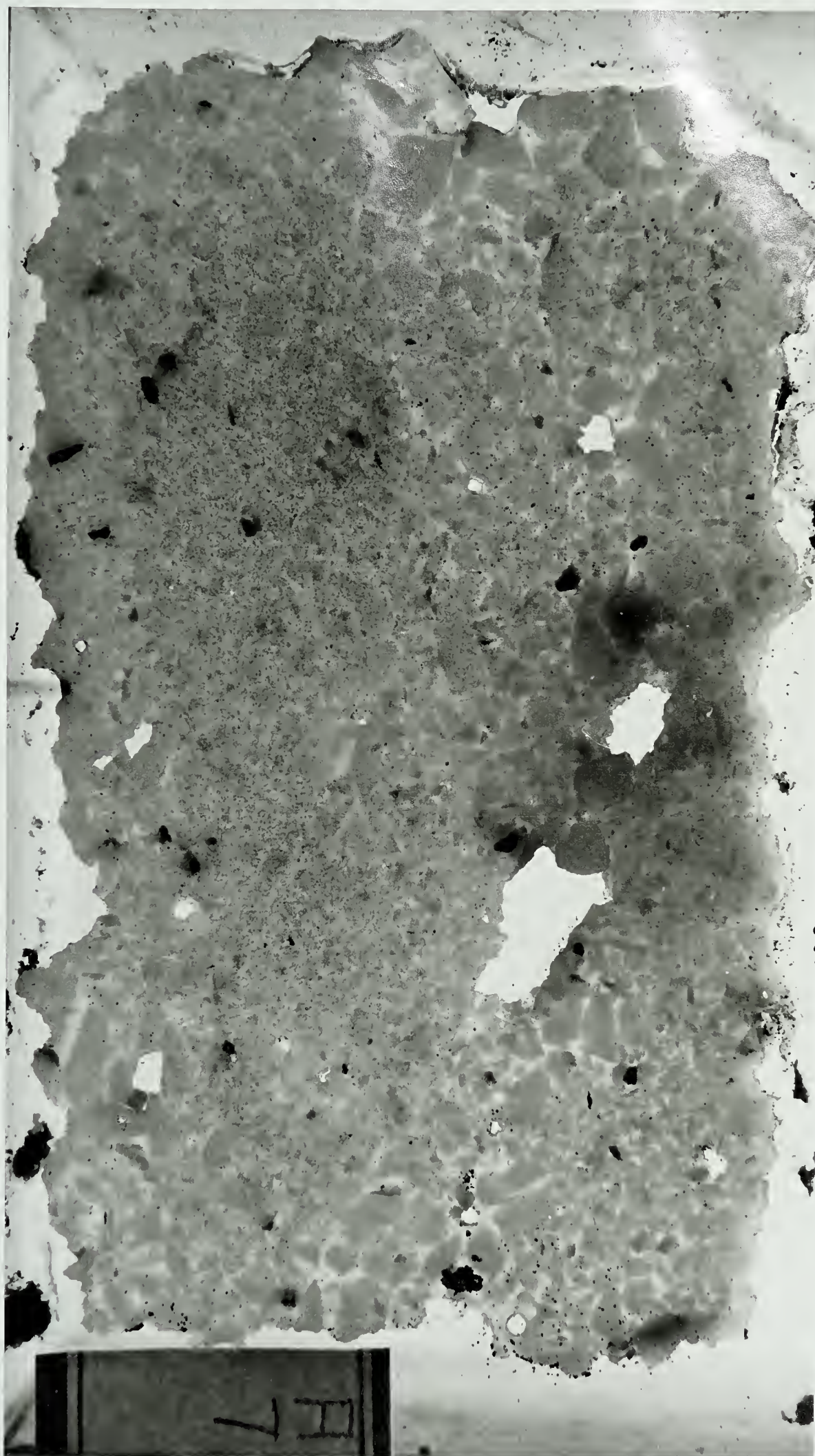
SECTION A - I



SECTION A- 5

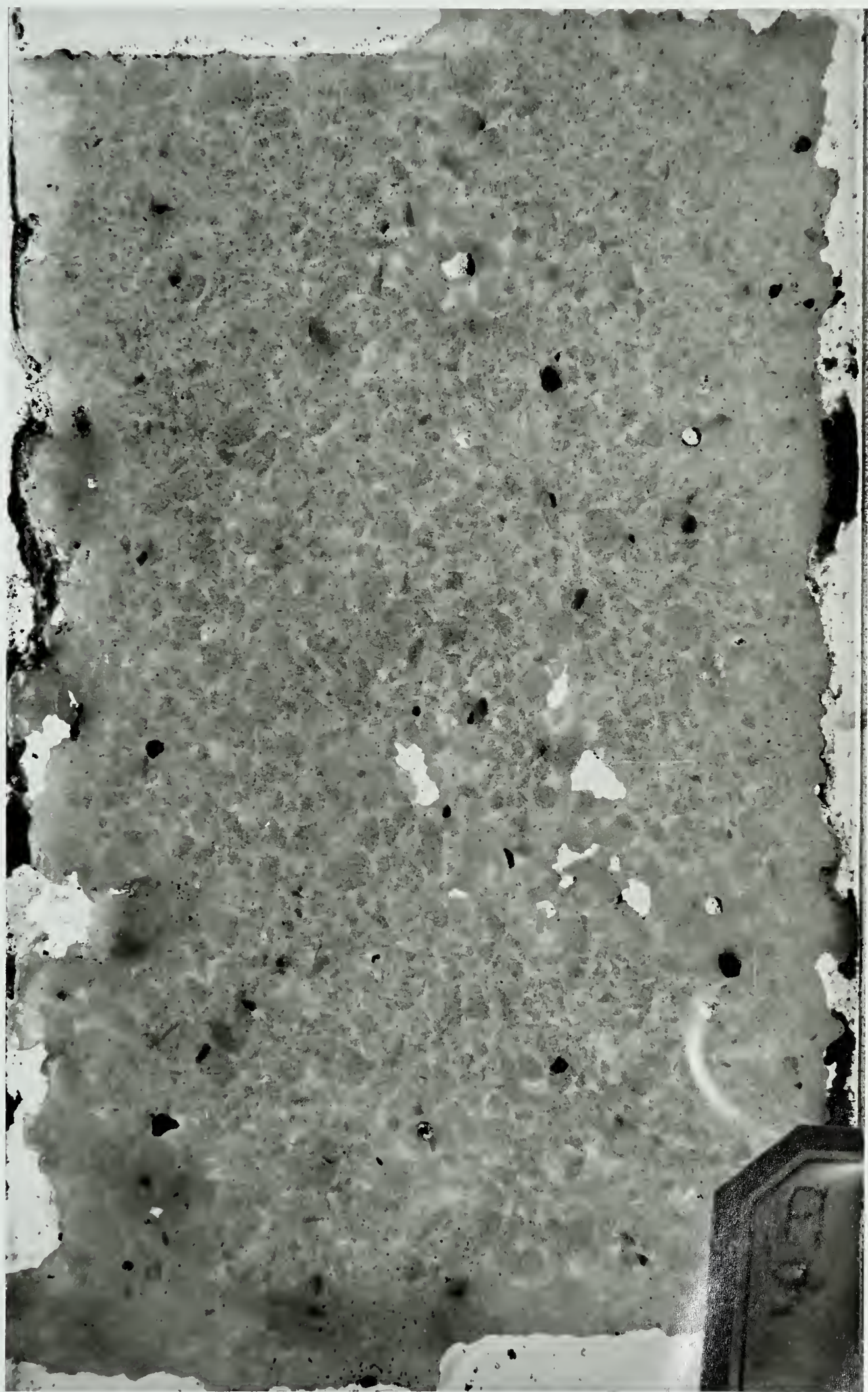


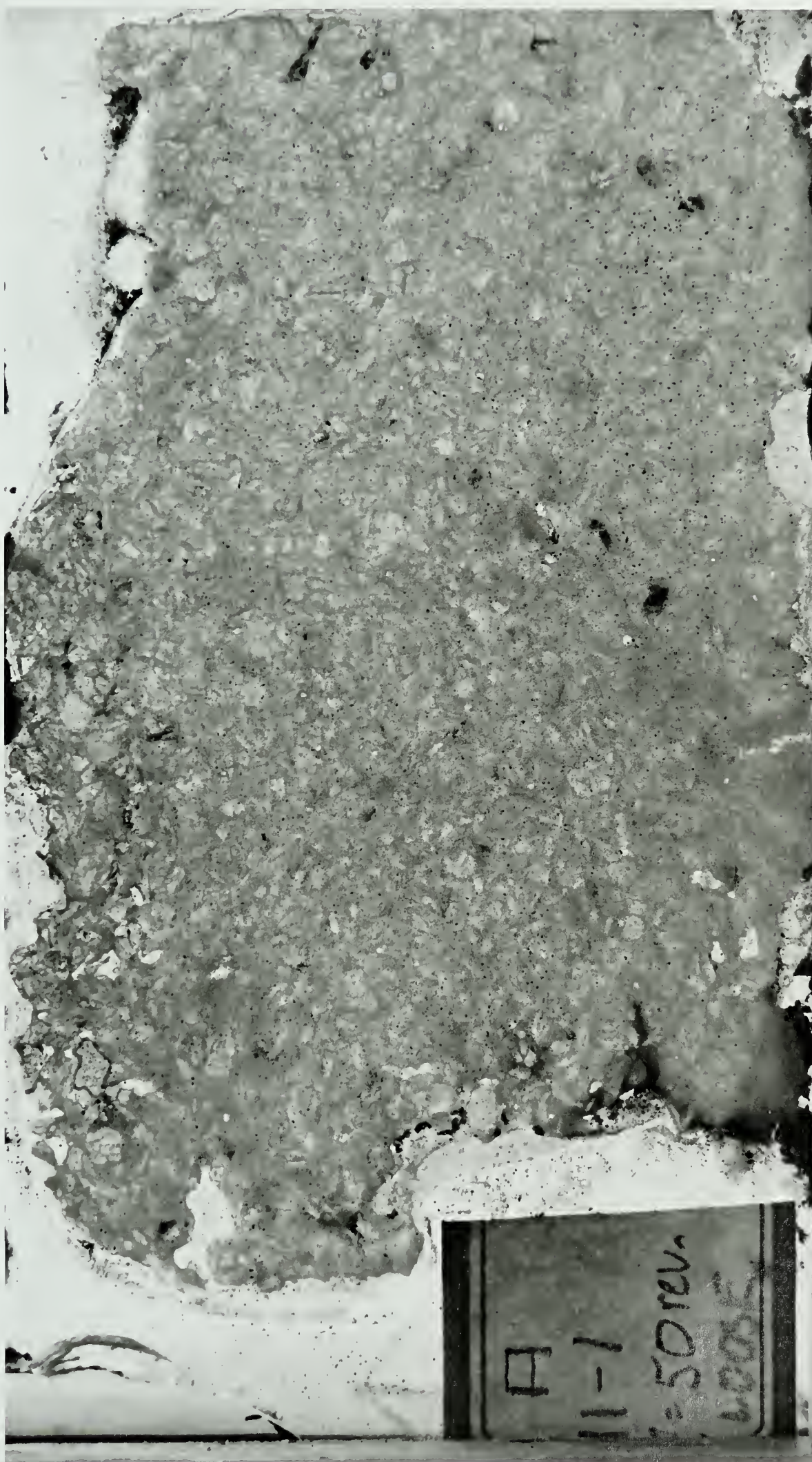
SECTION A -6

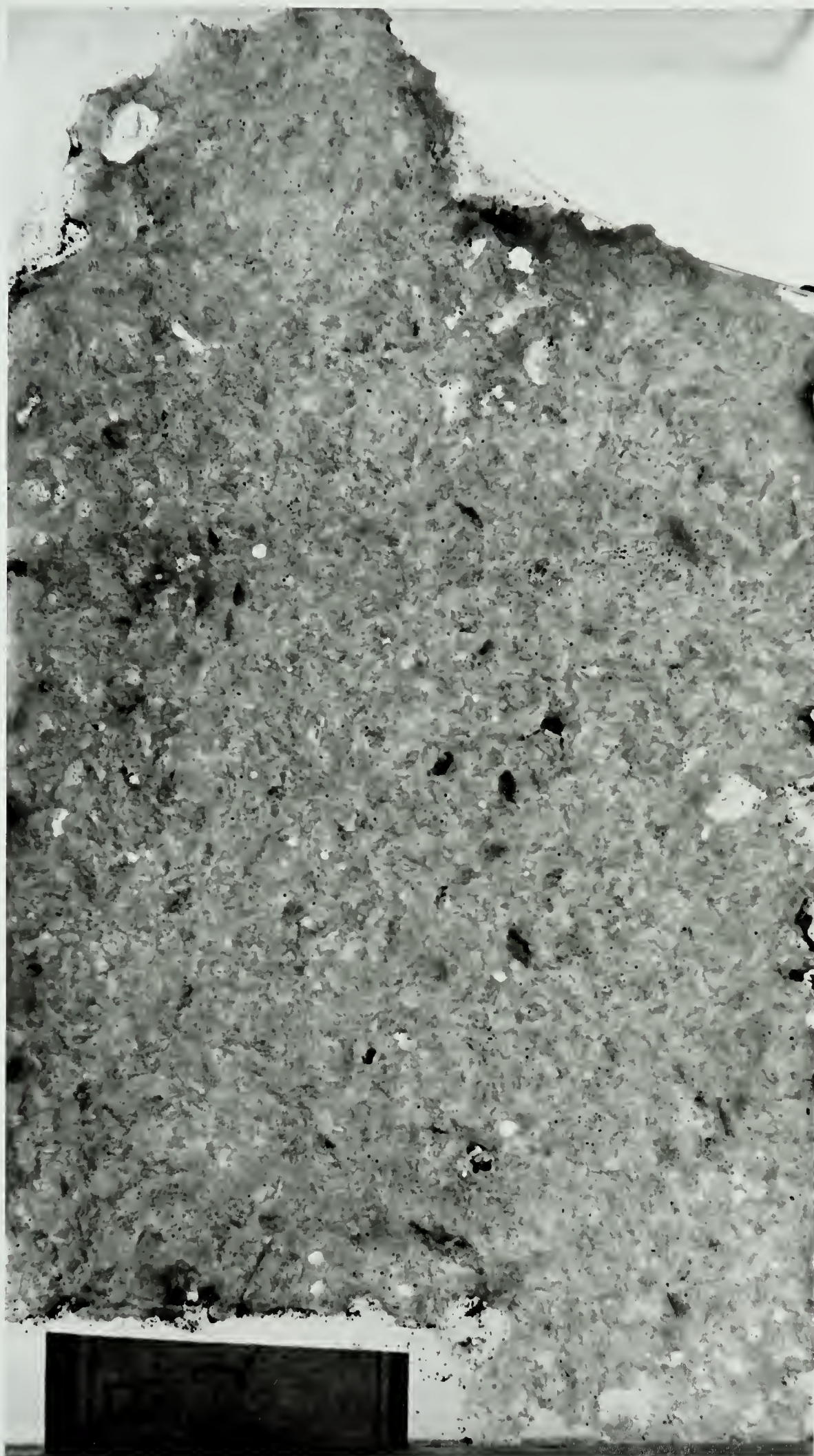


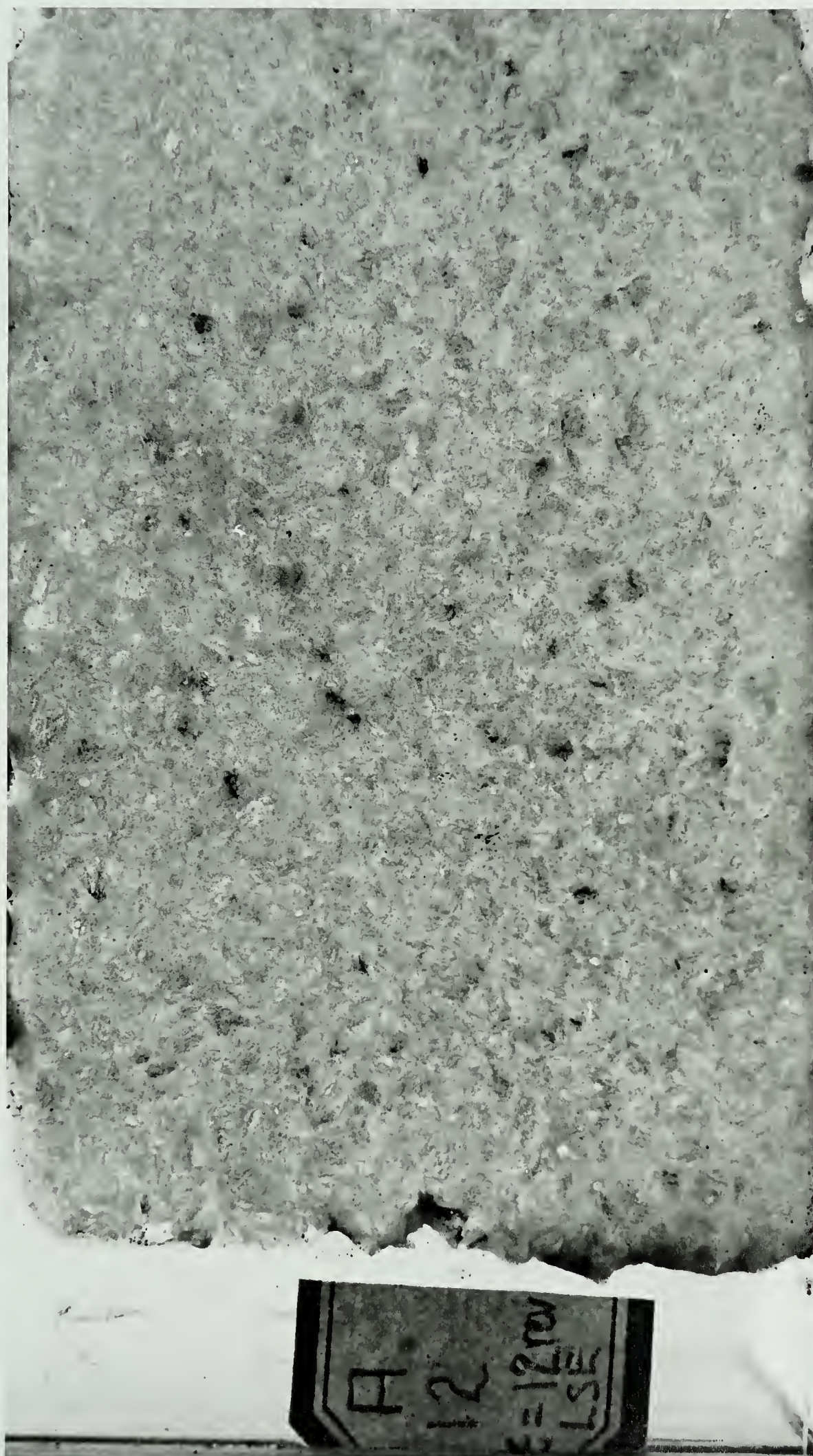


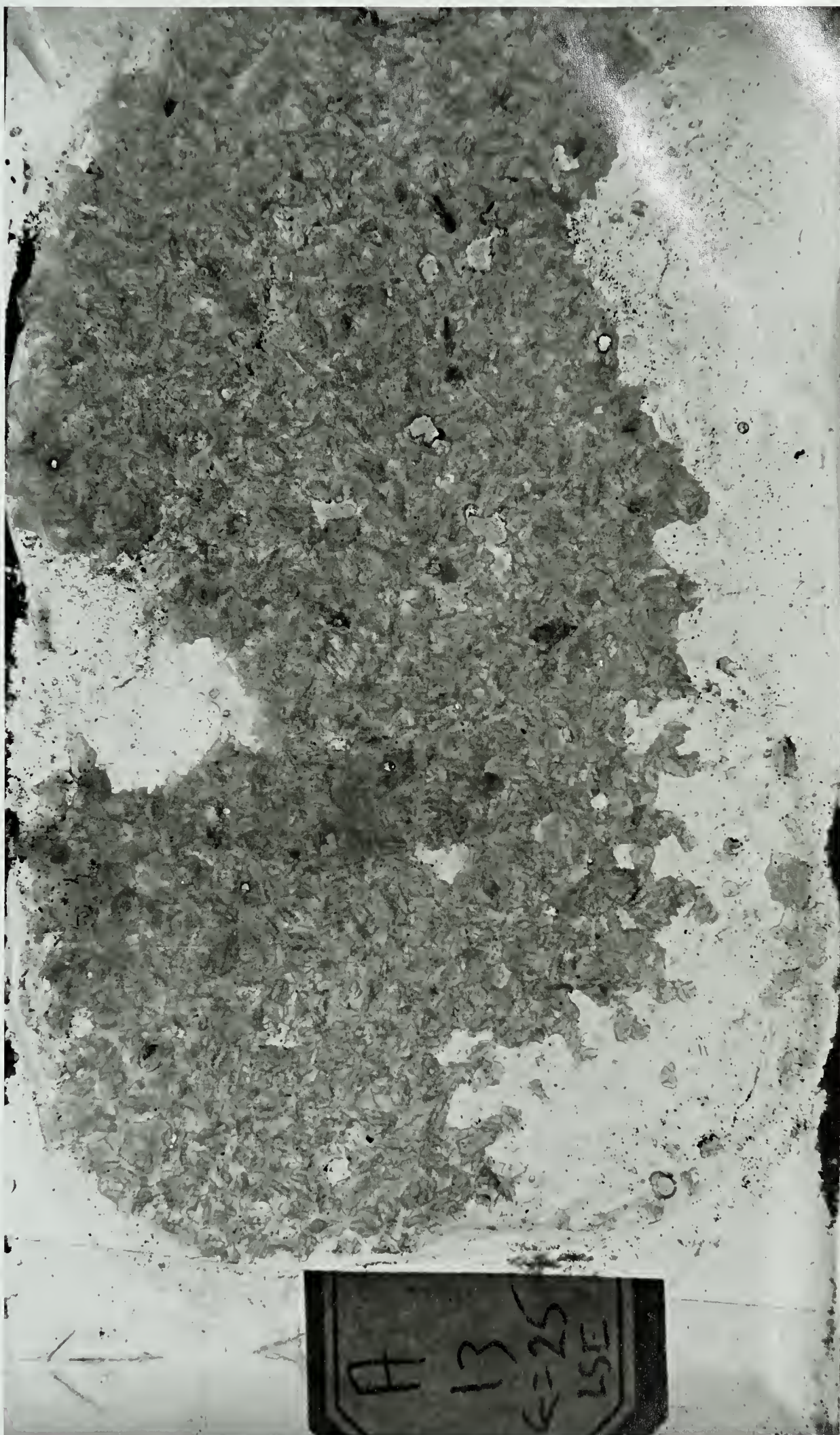
SECTION A -8

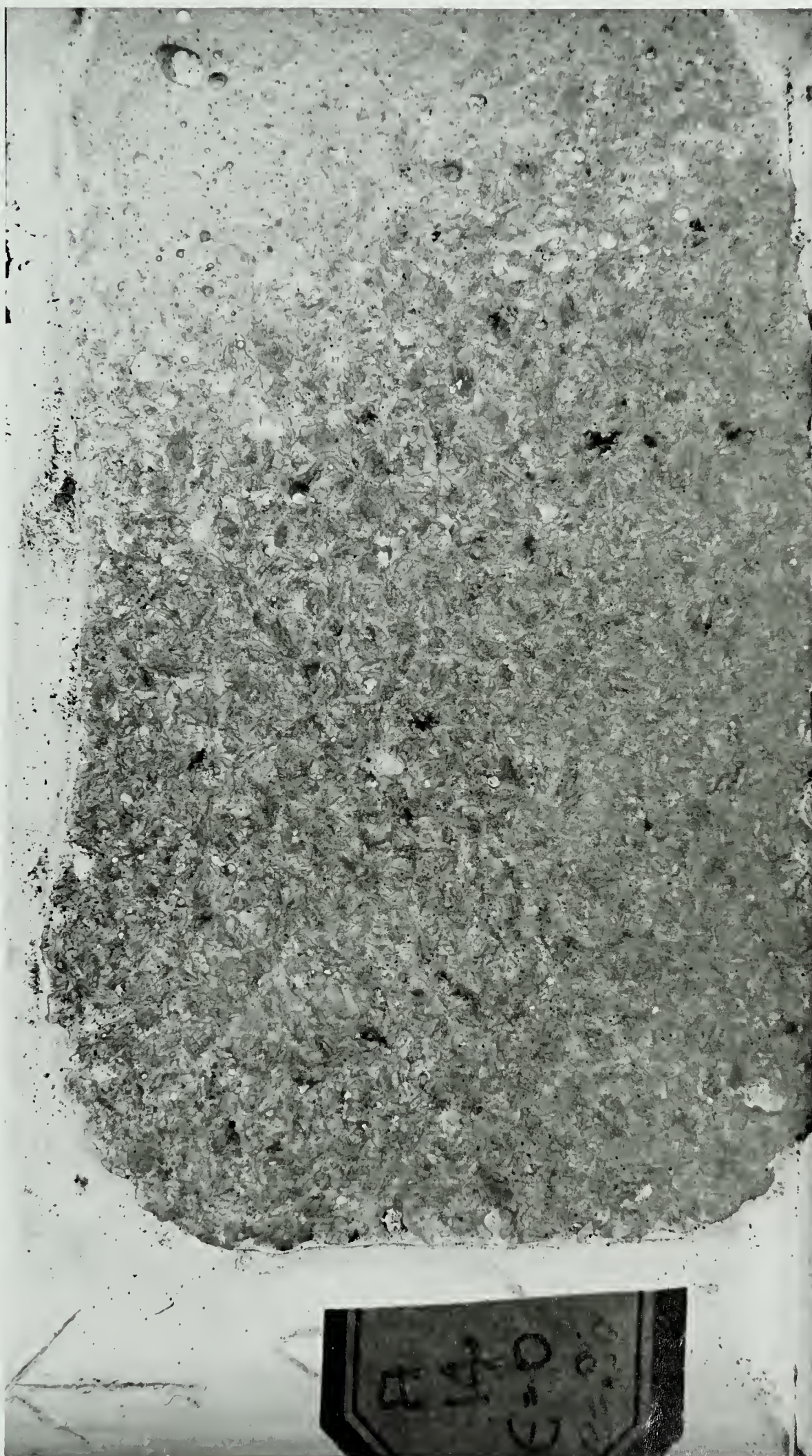


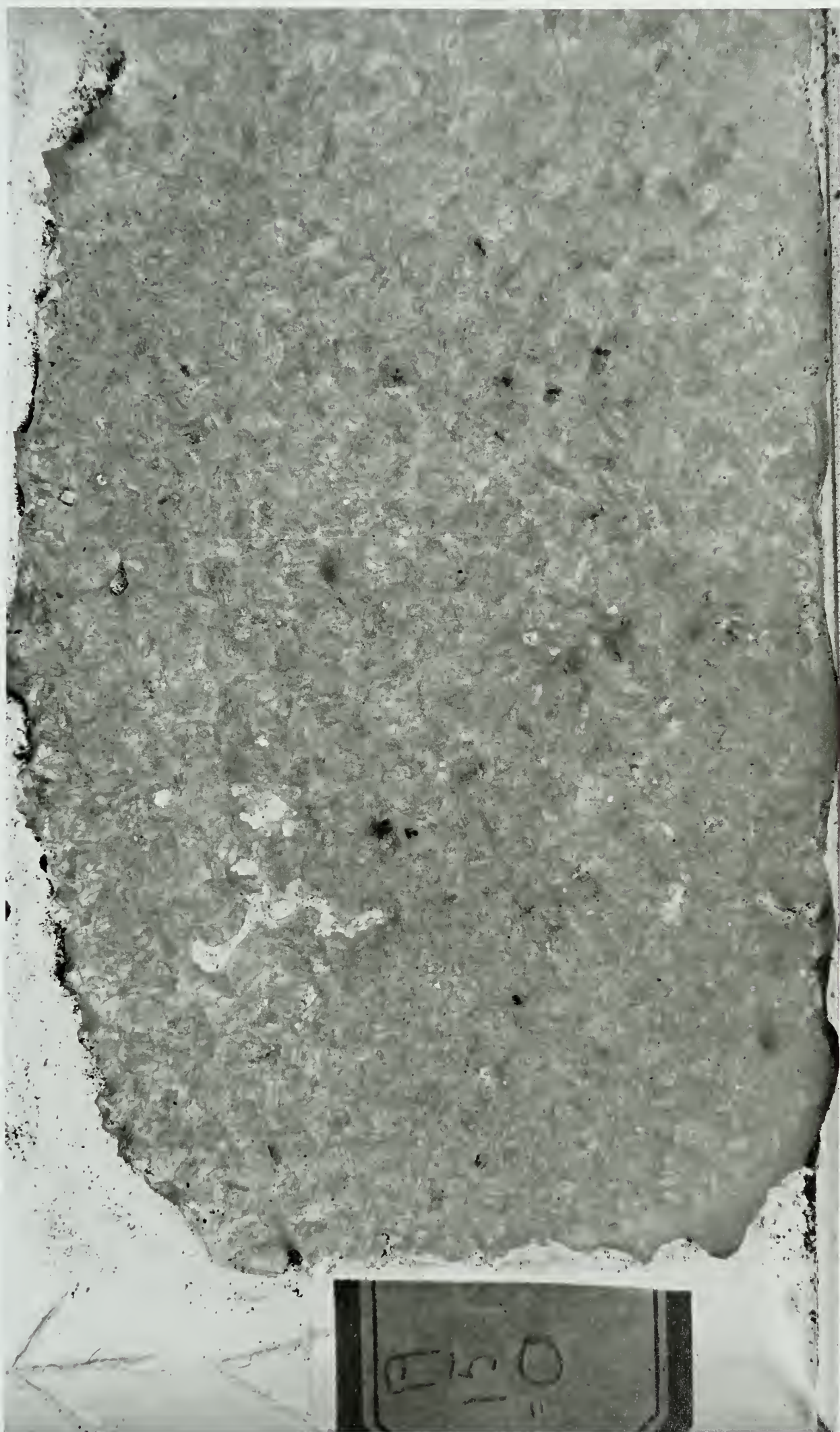




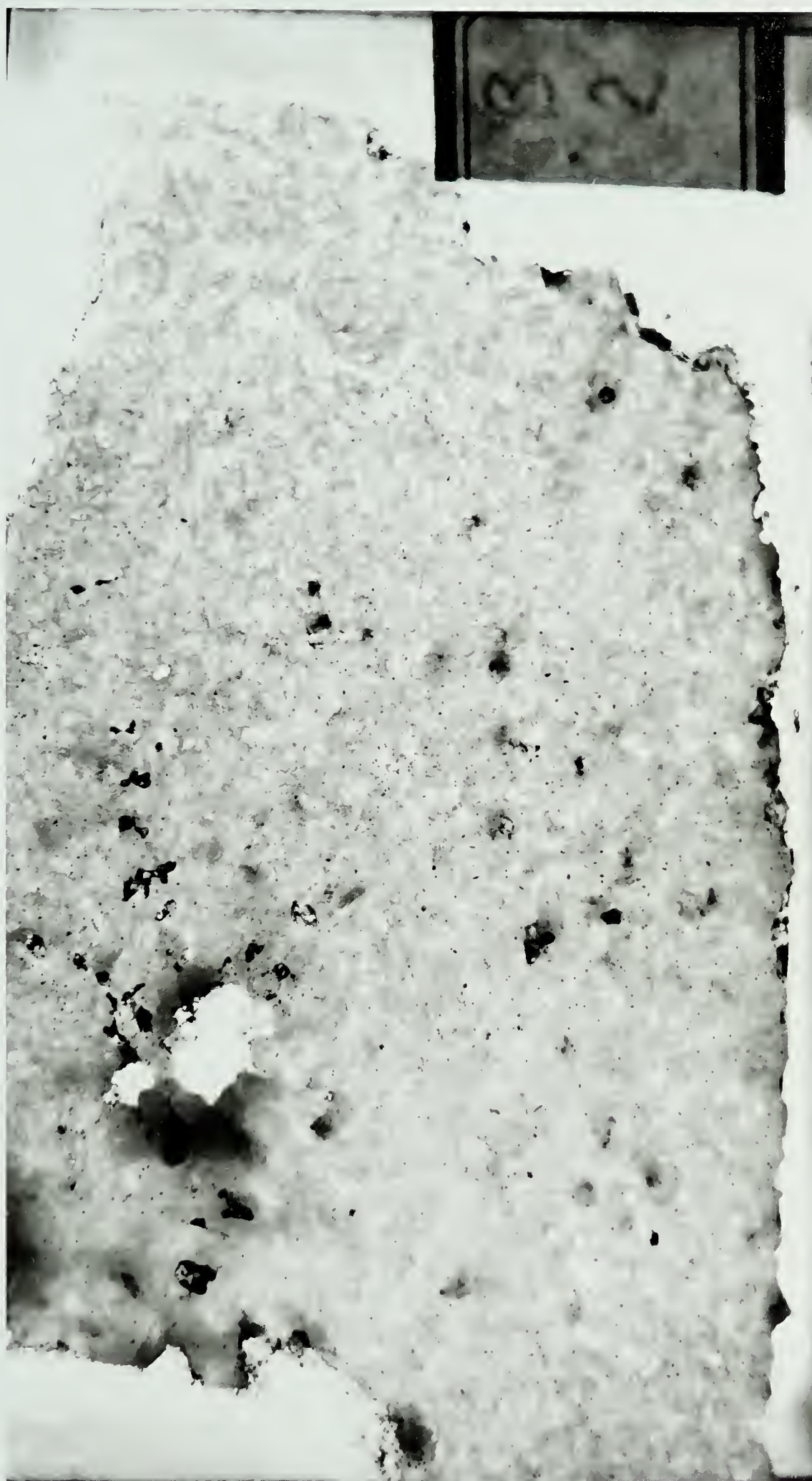


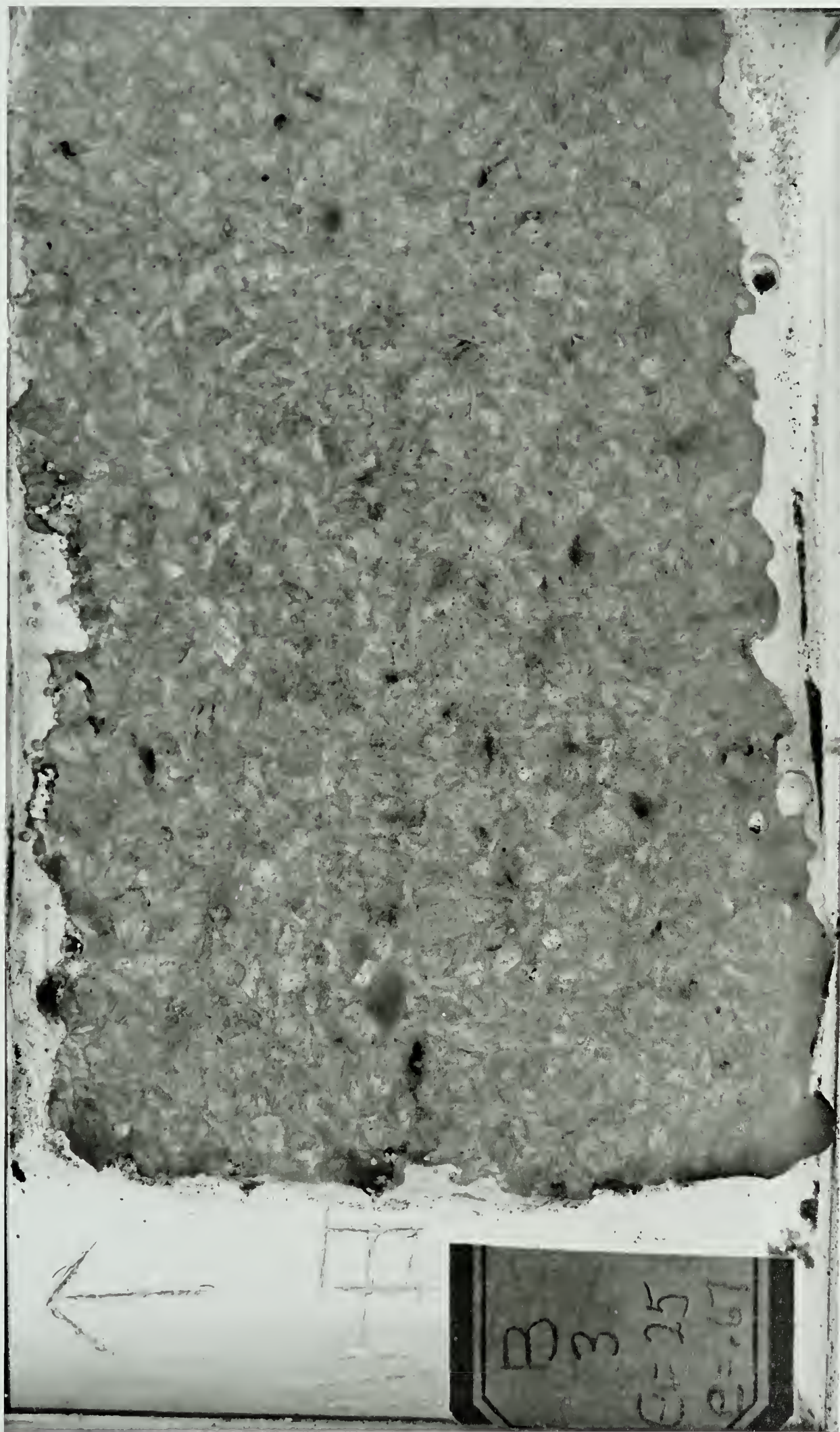


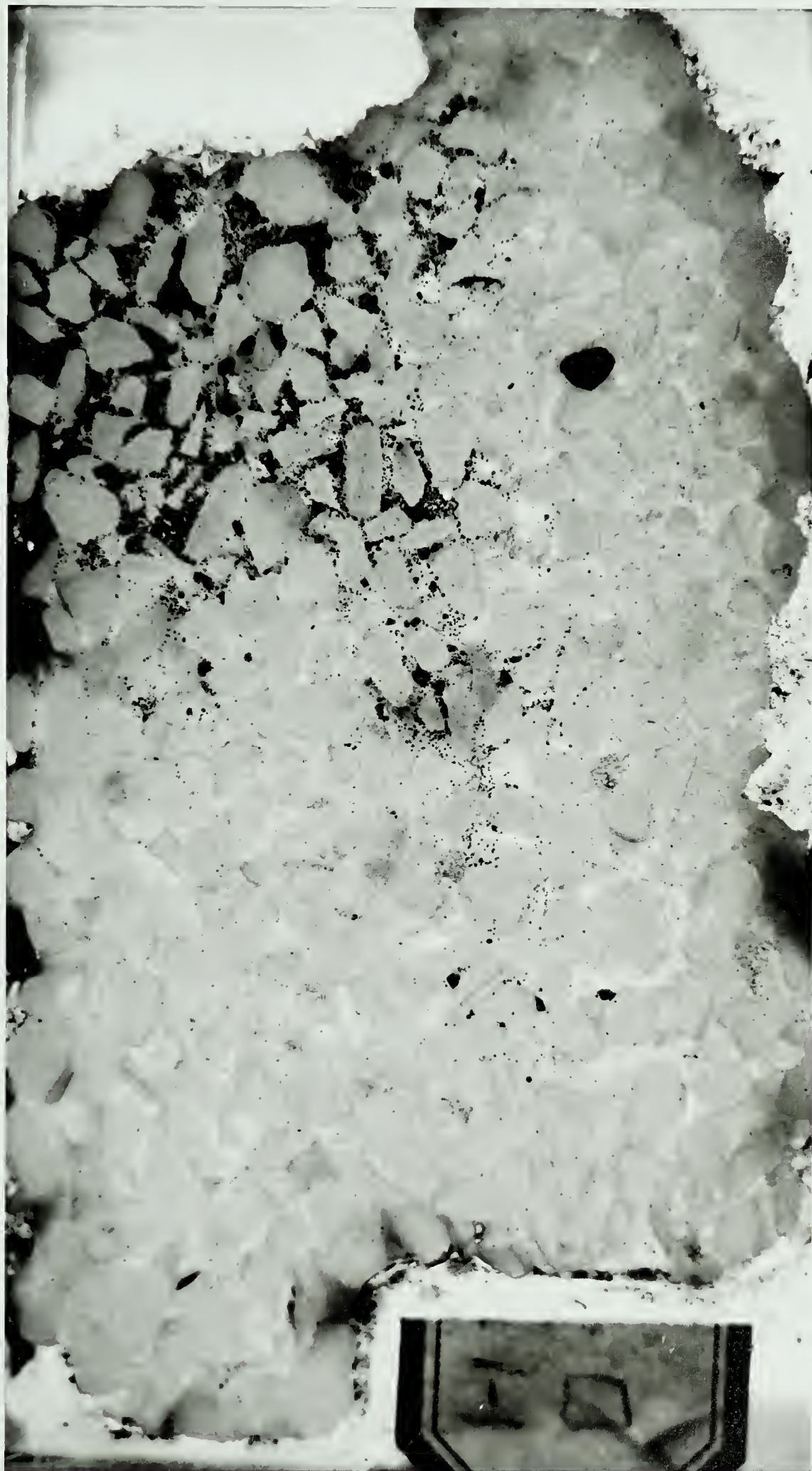


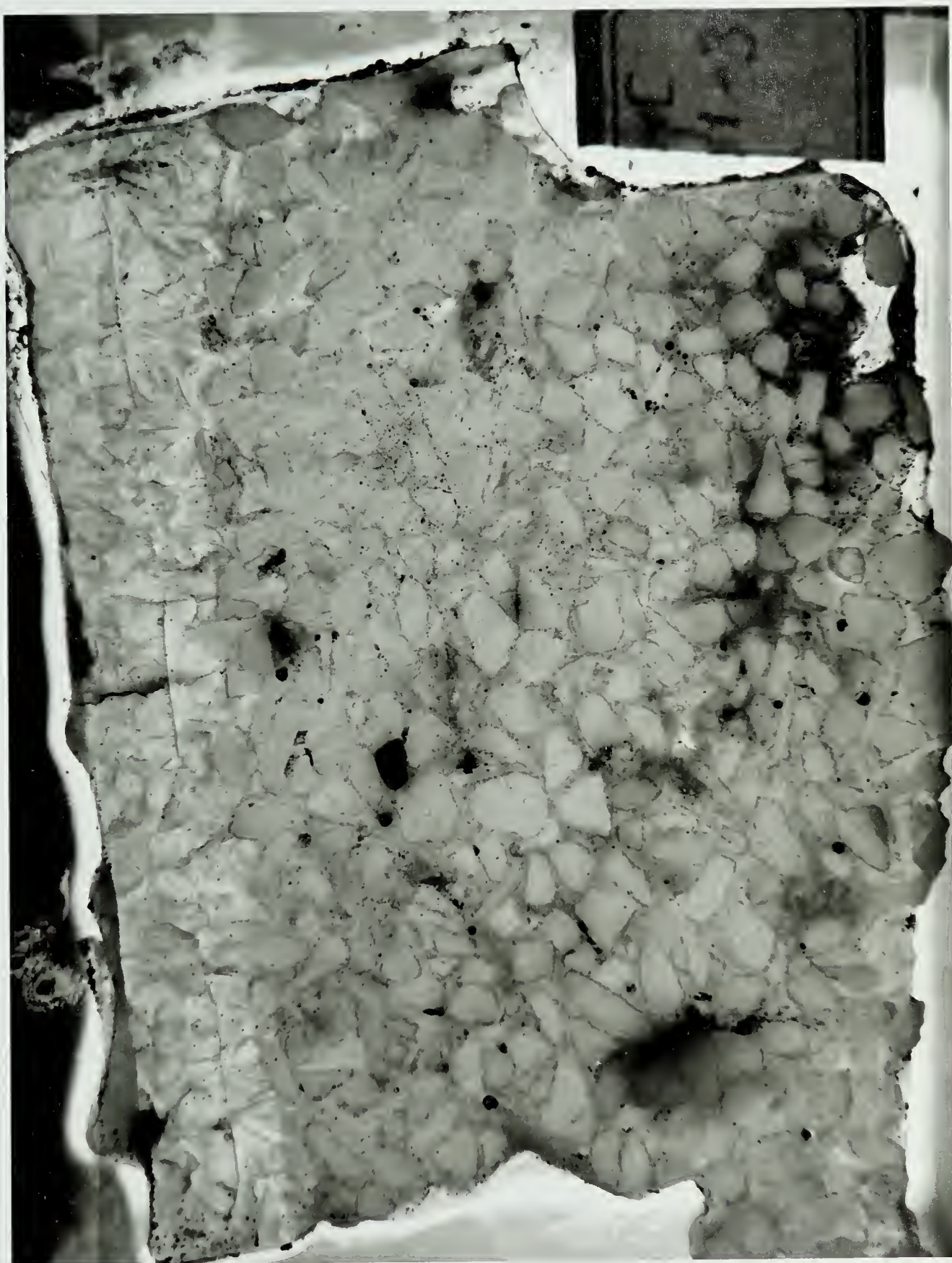


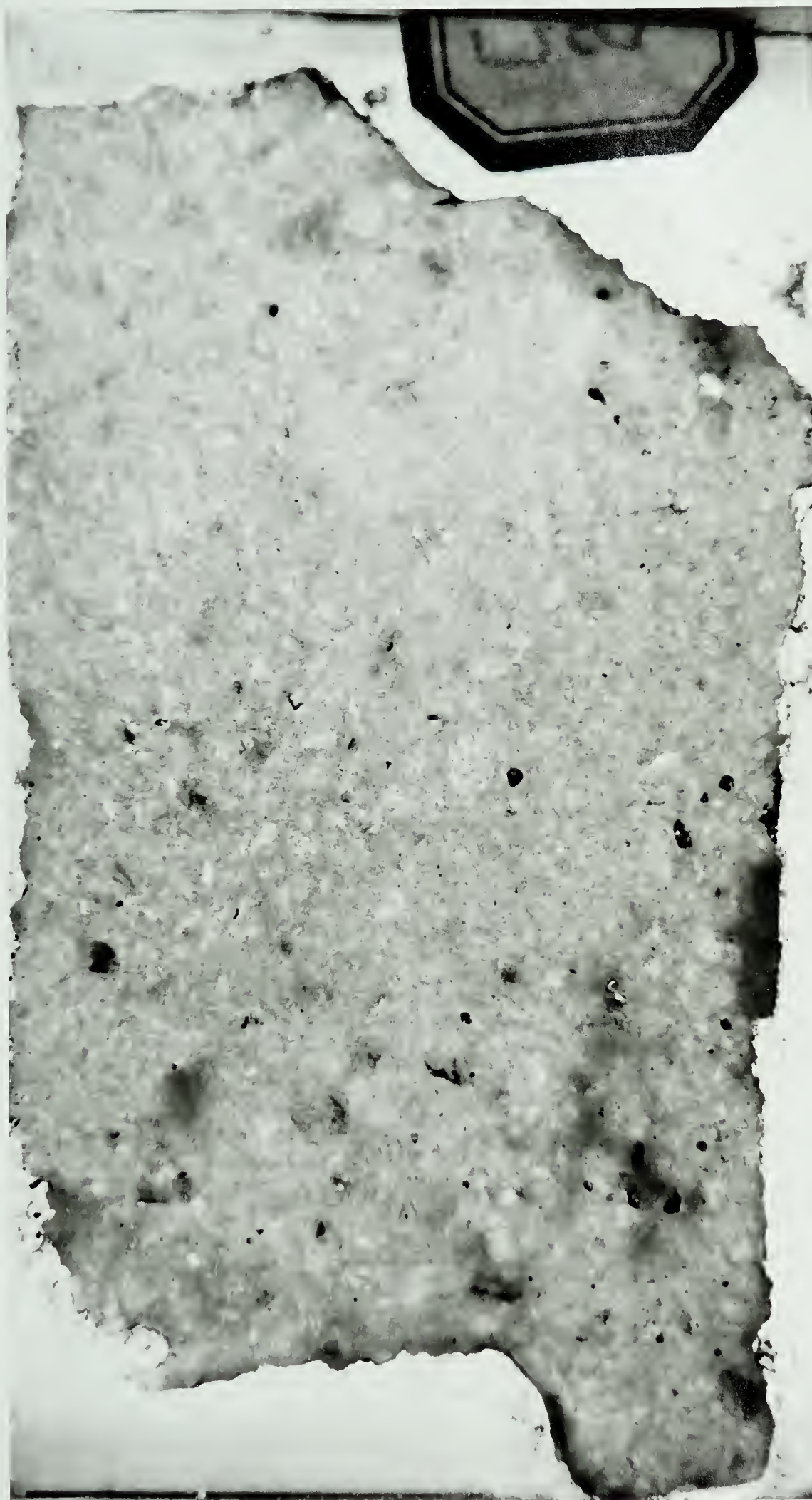


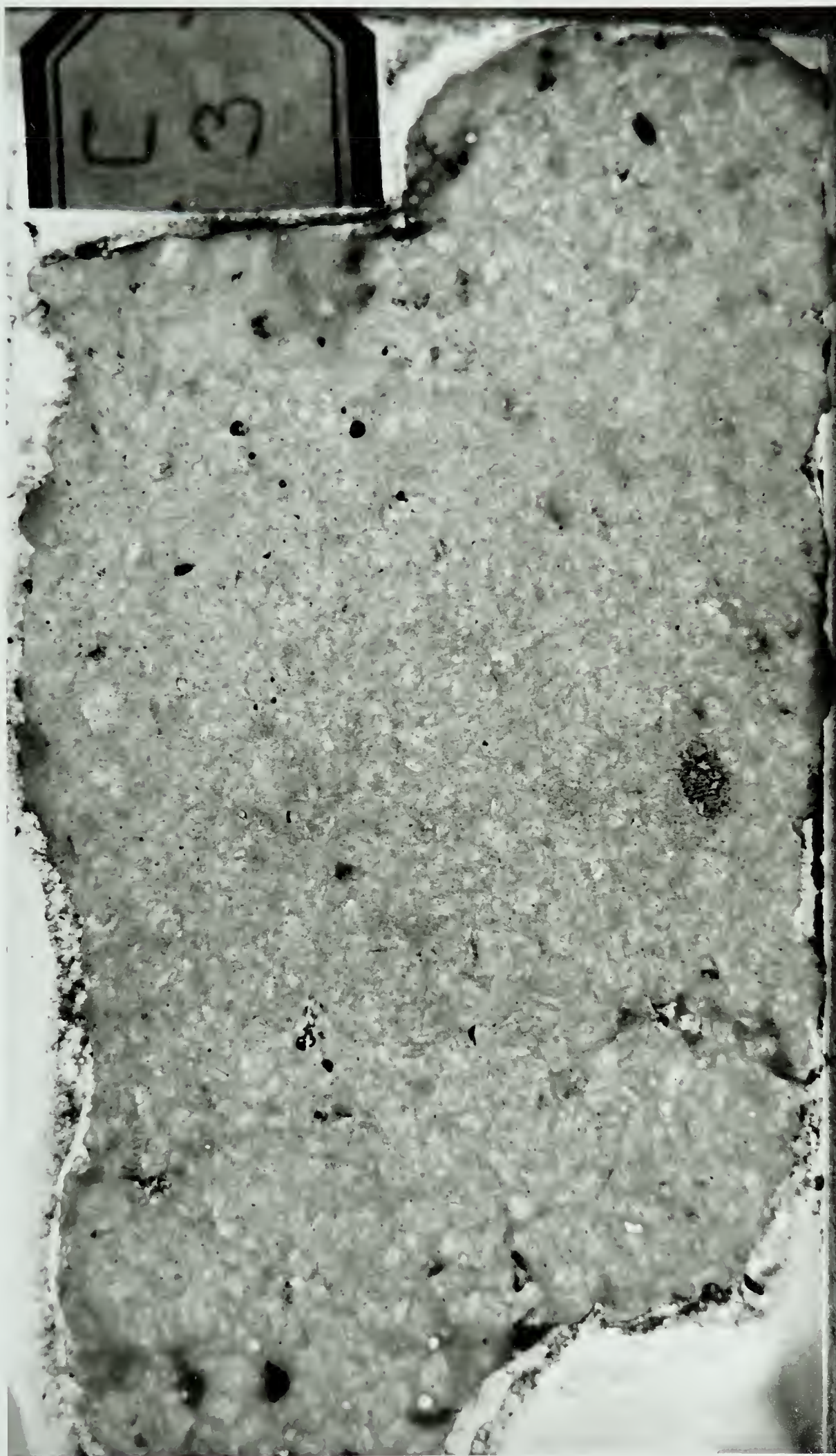




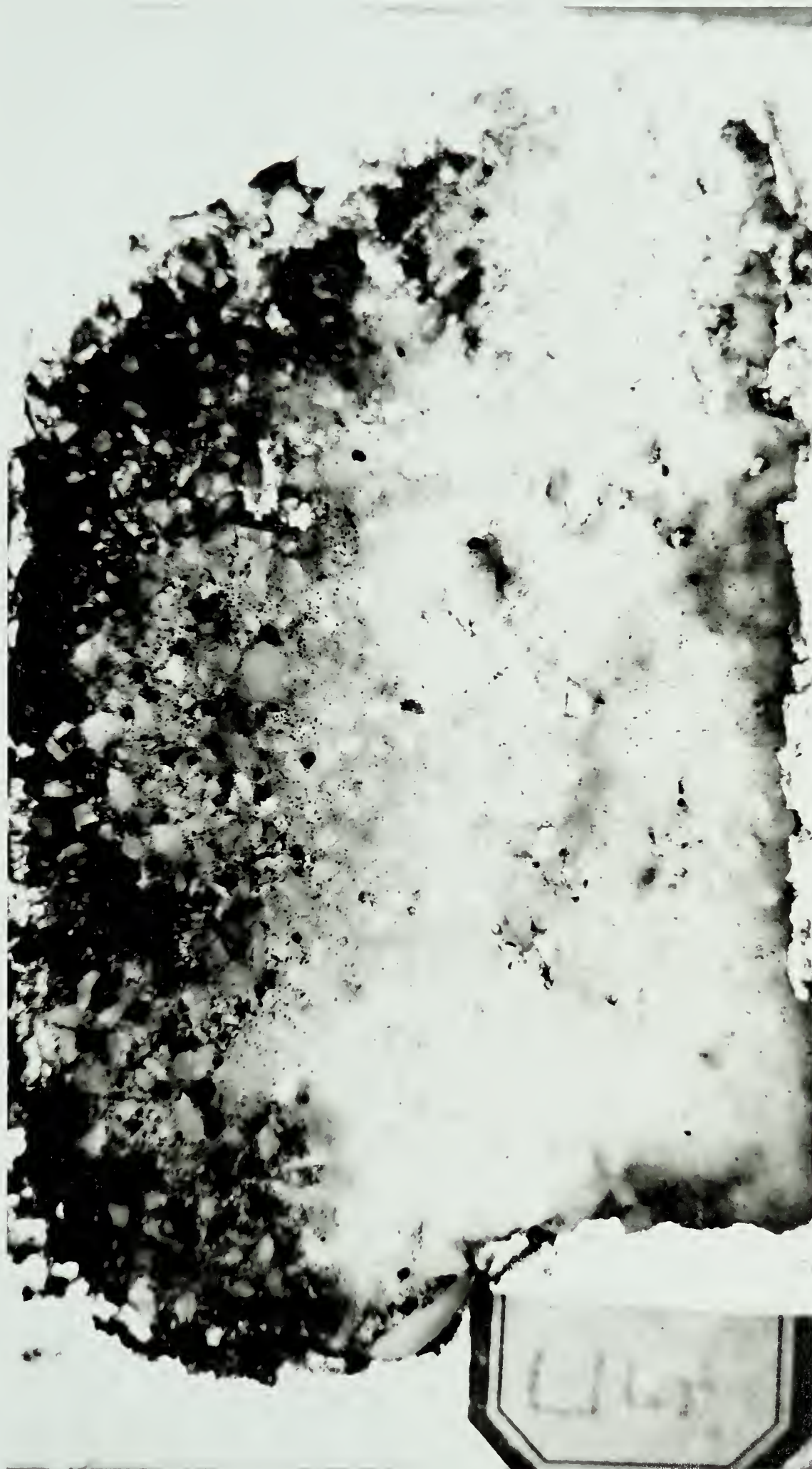


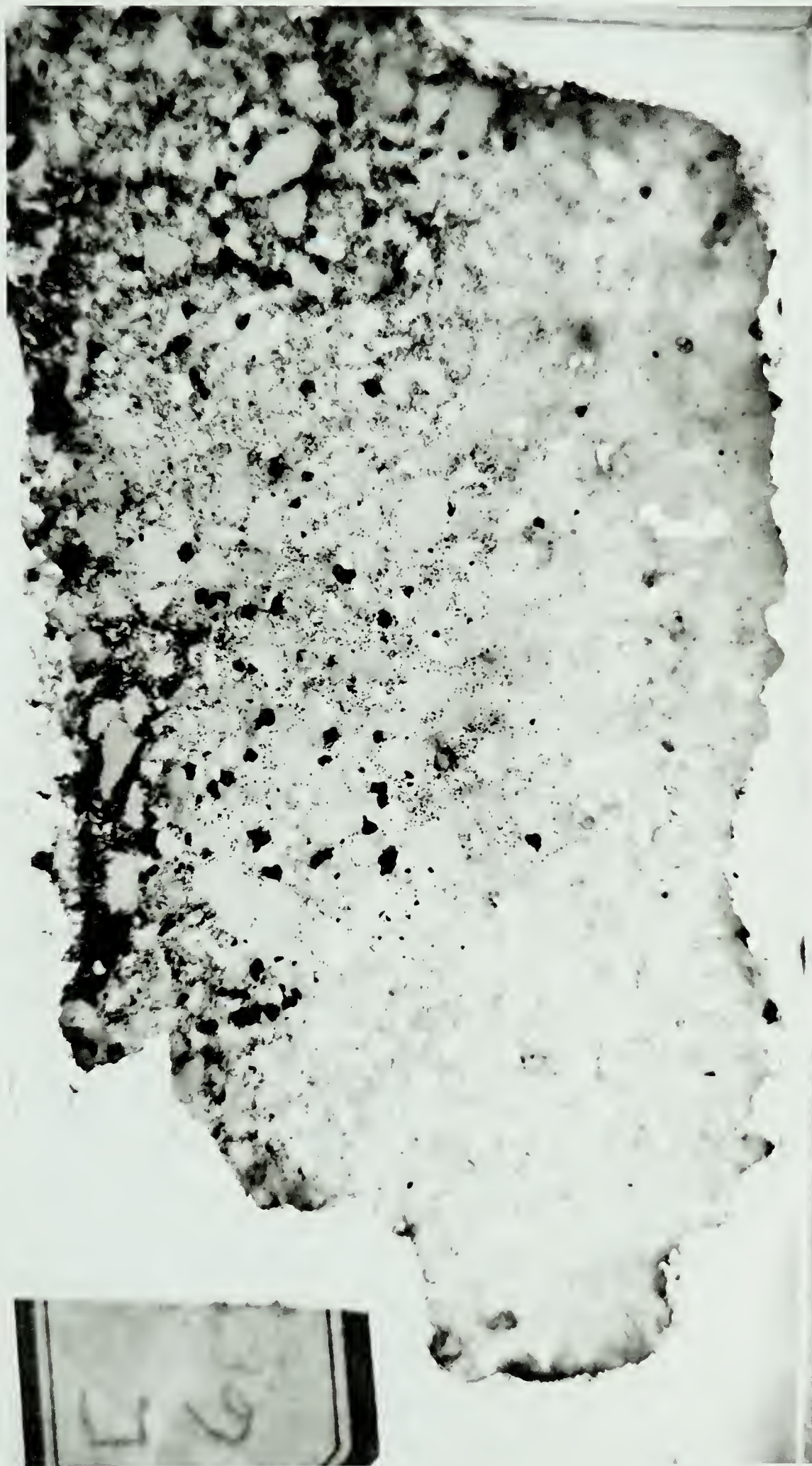




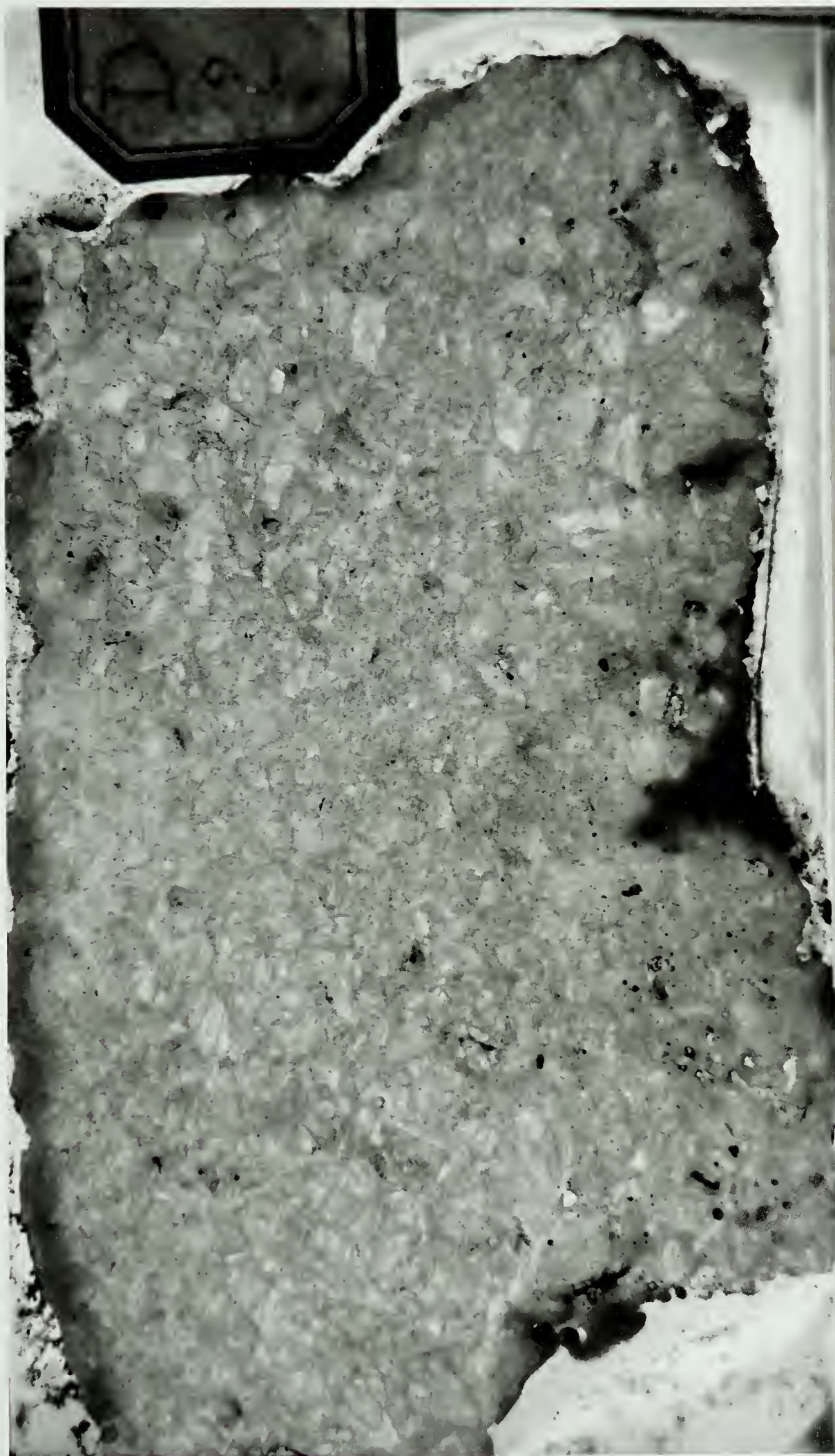


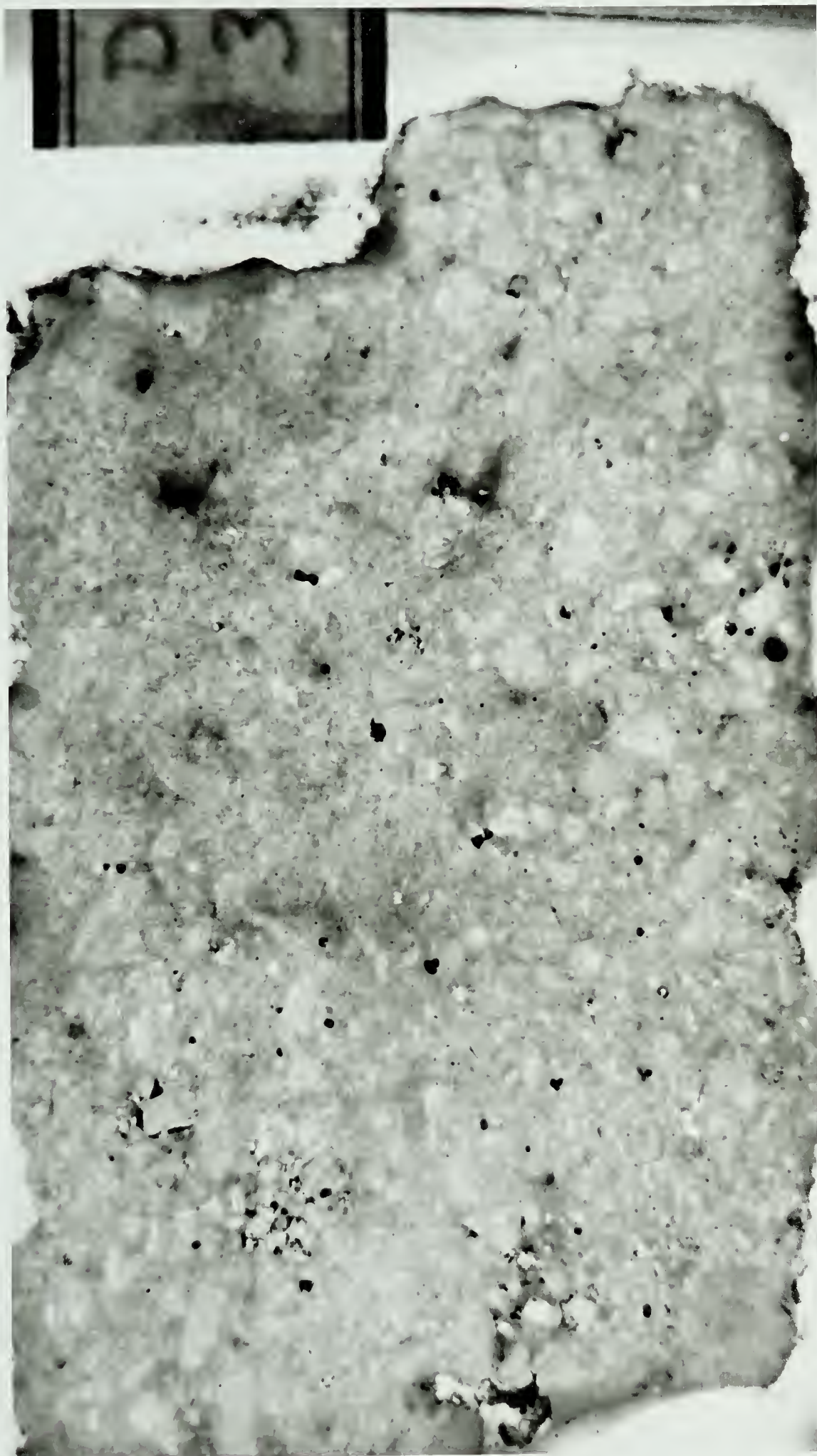


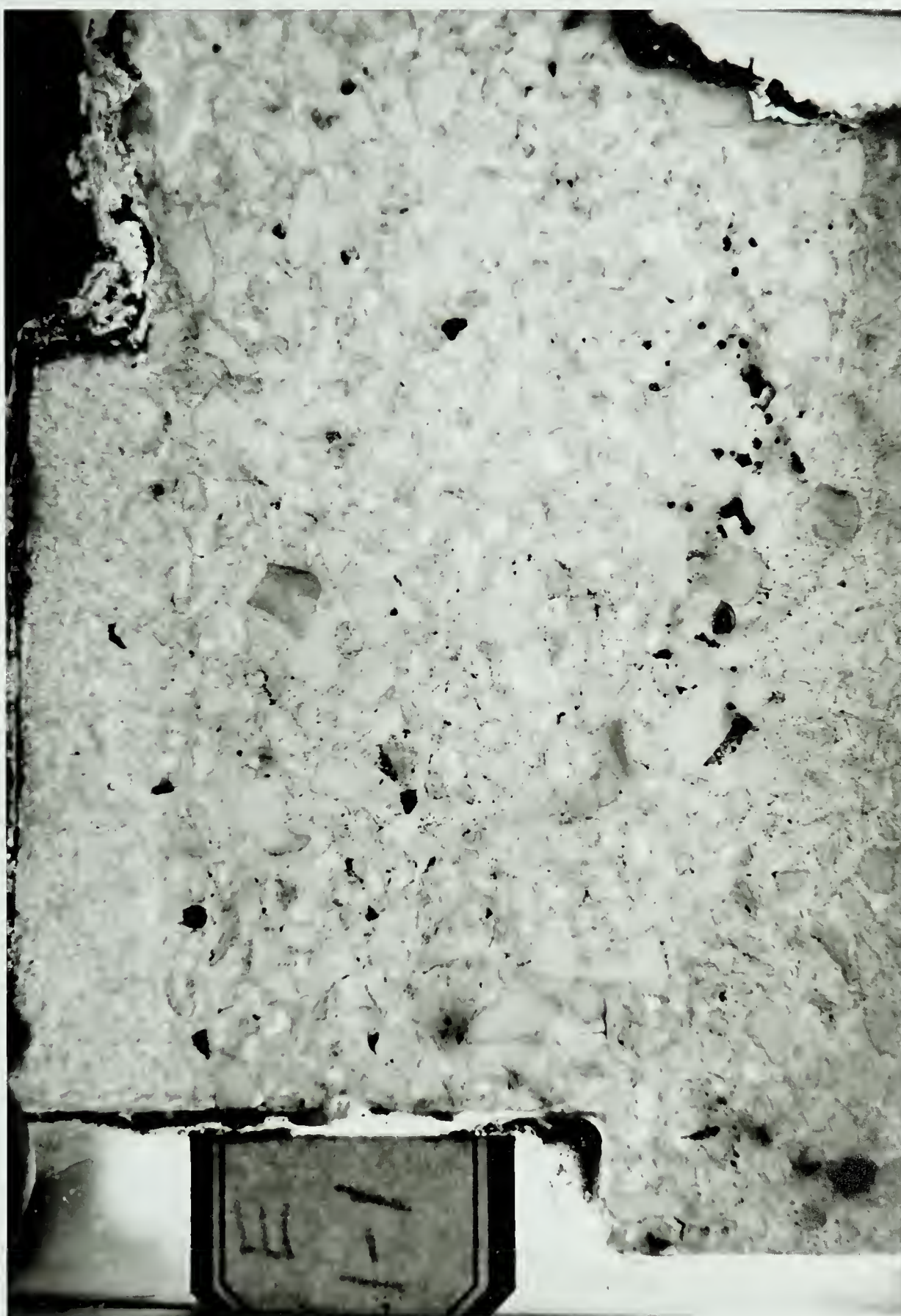


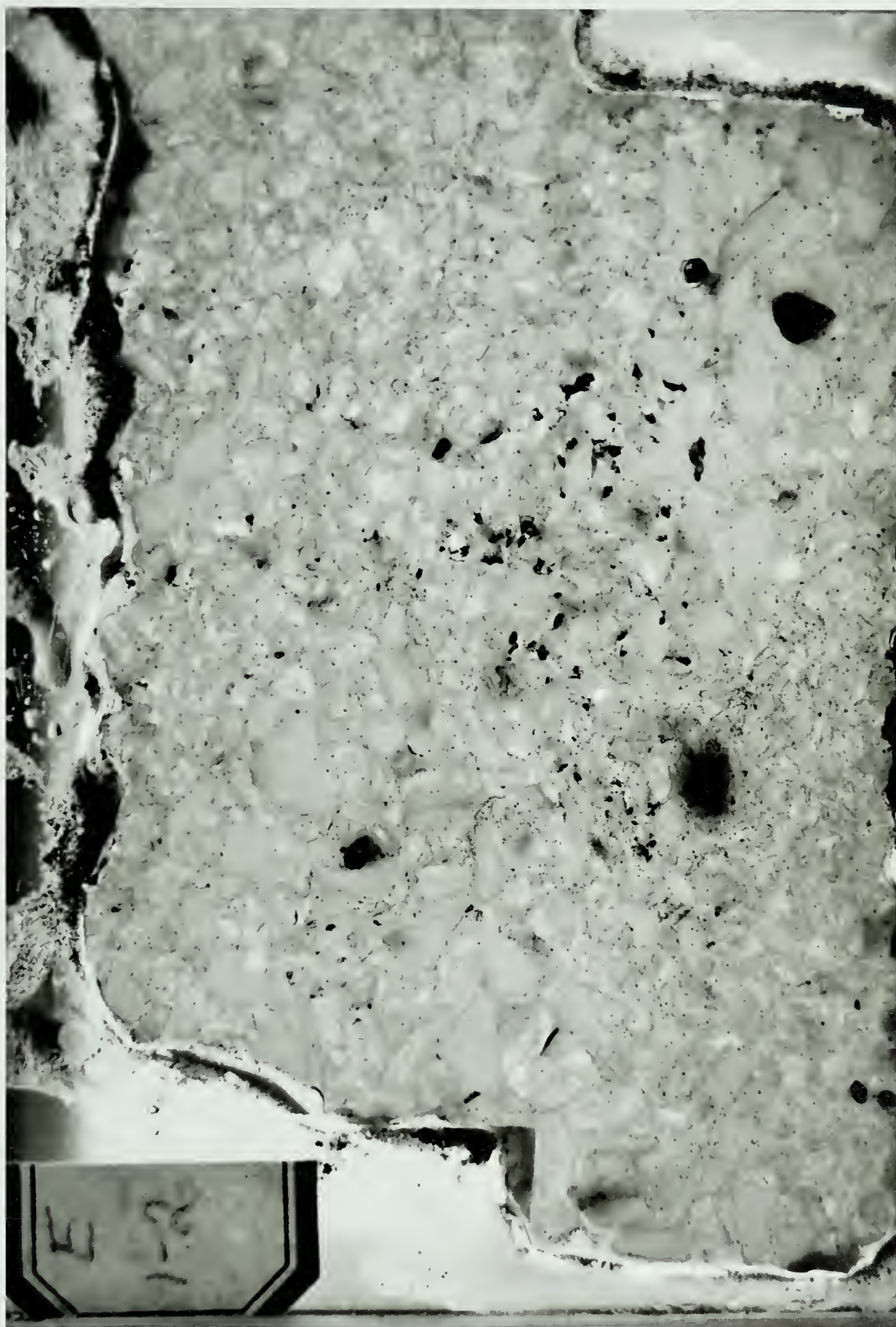


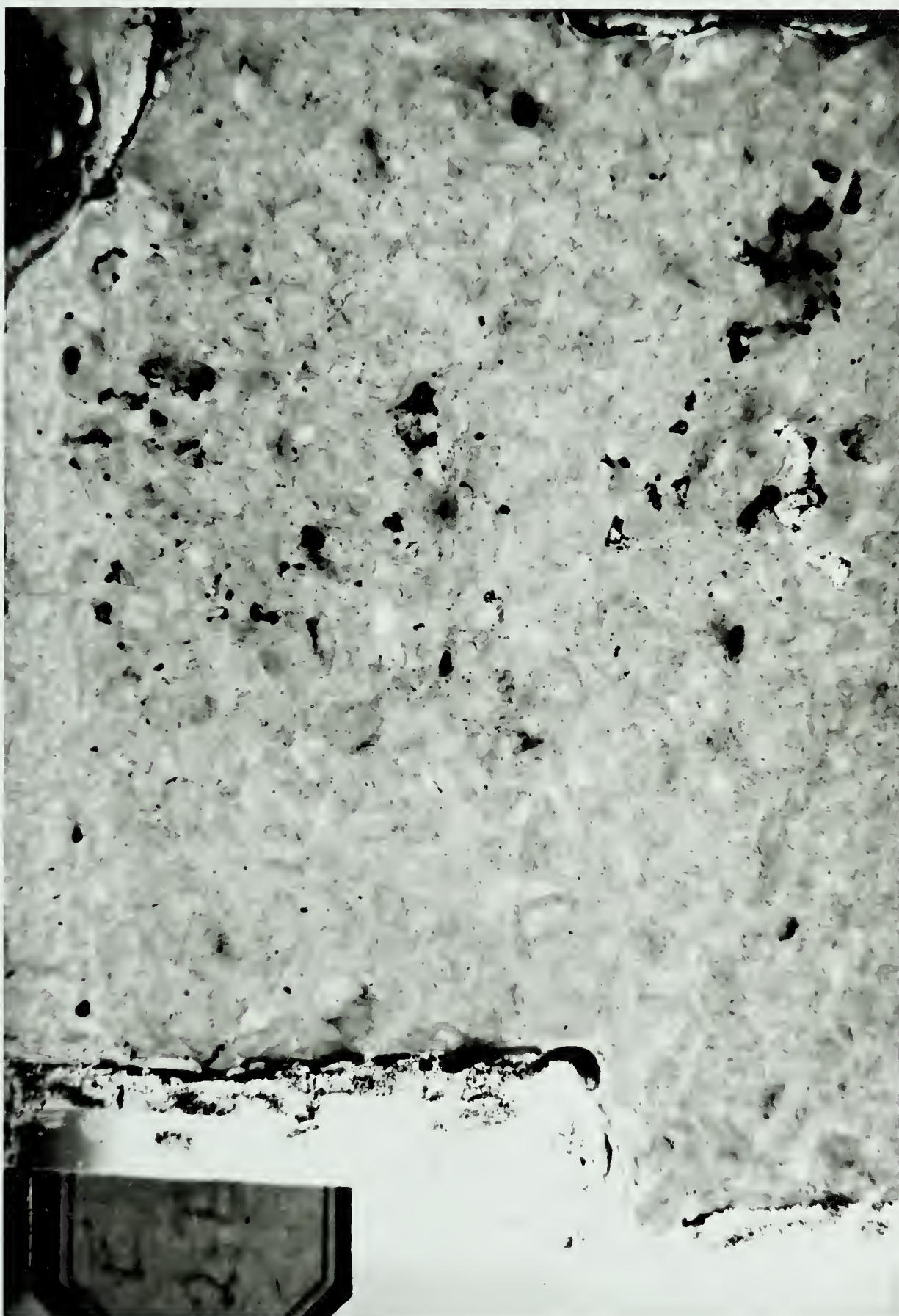


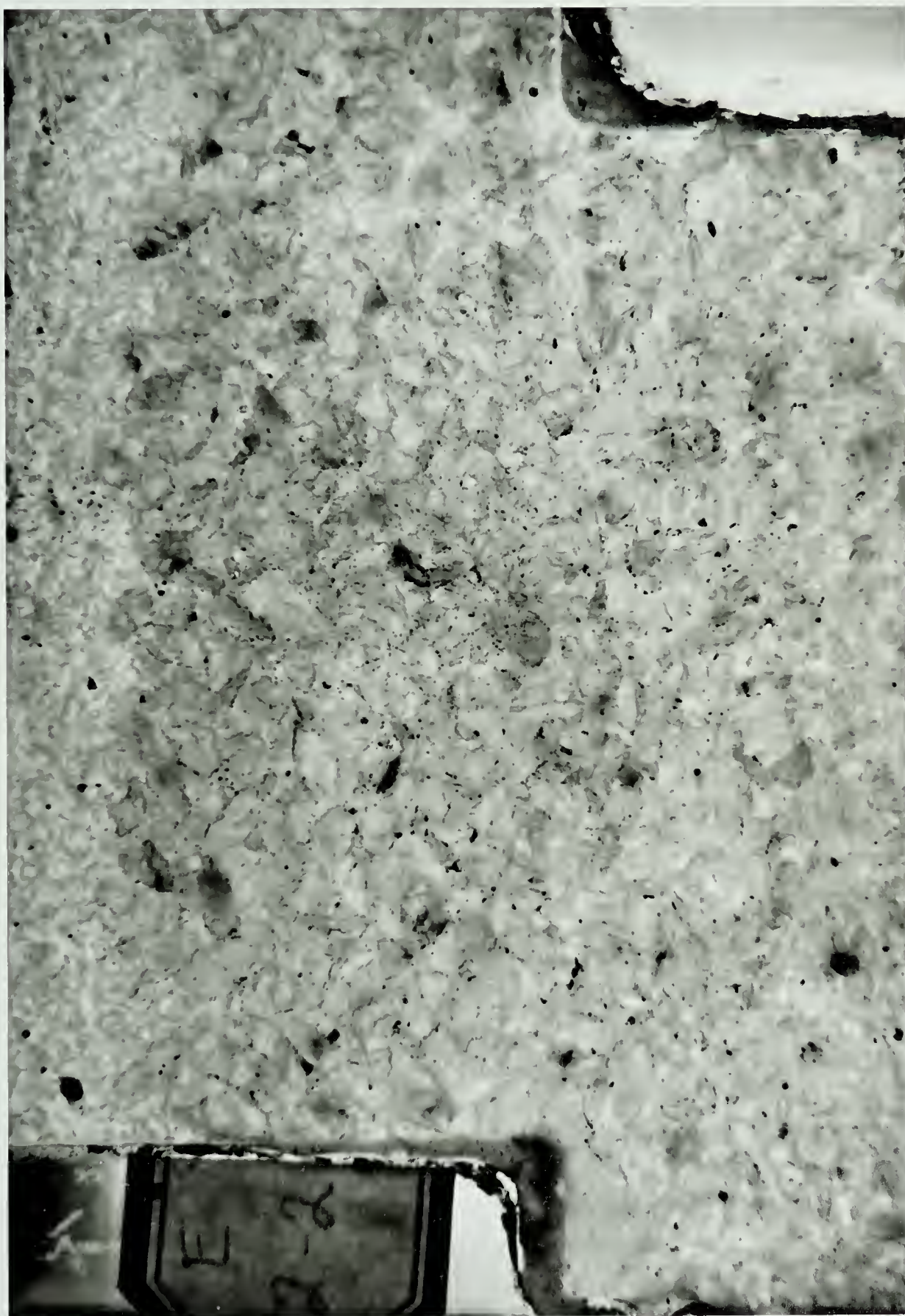


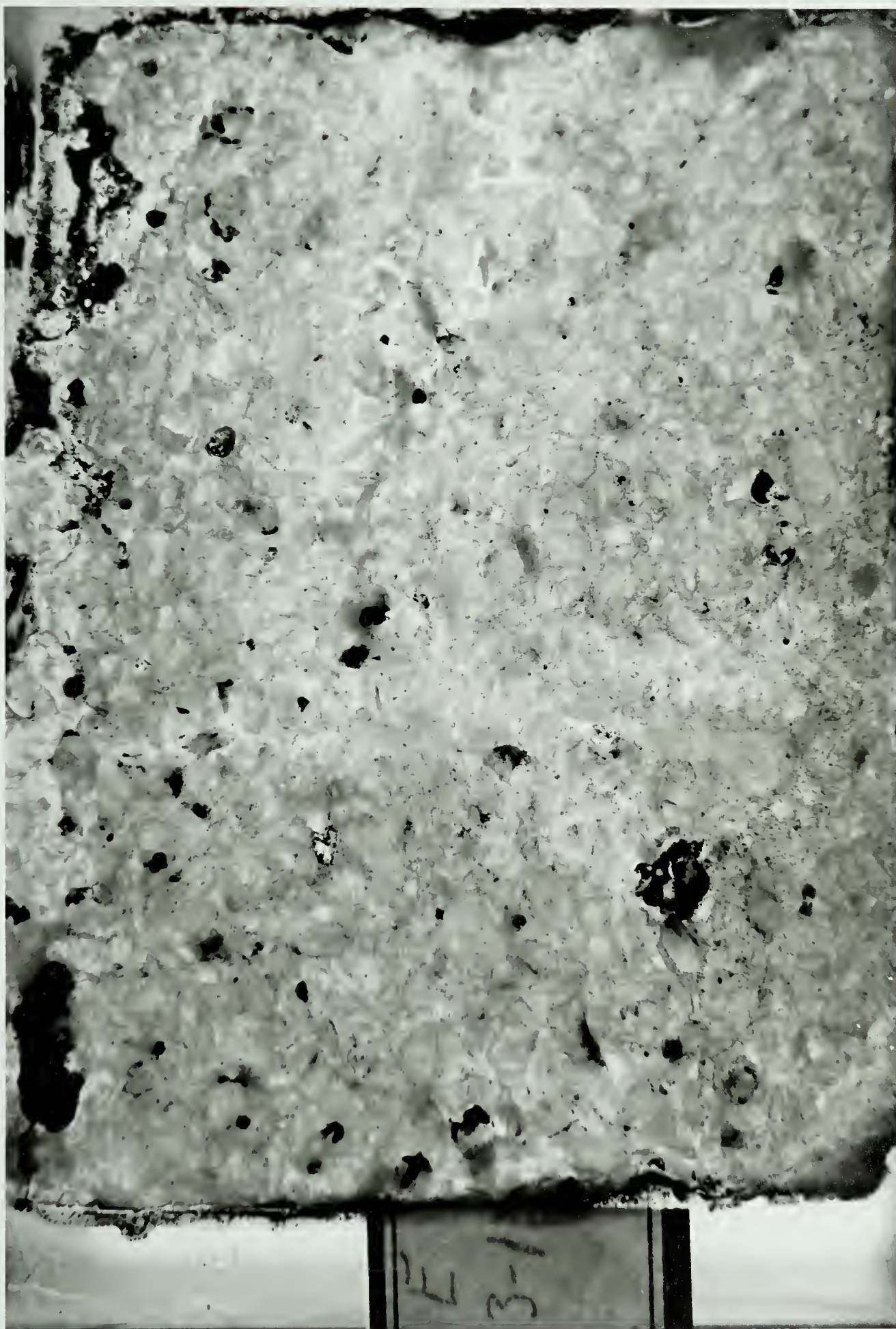


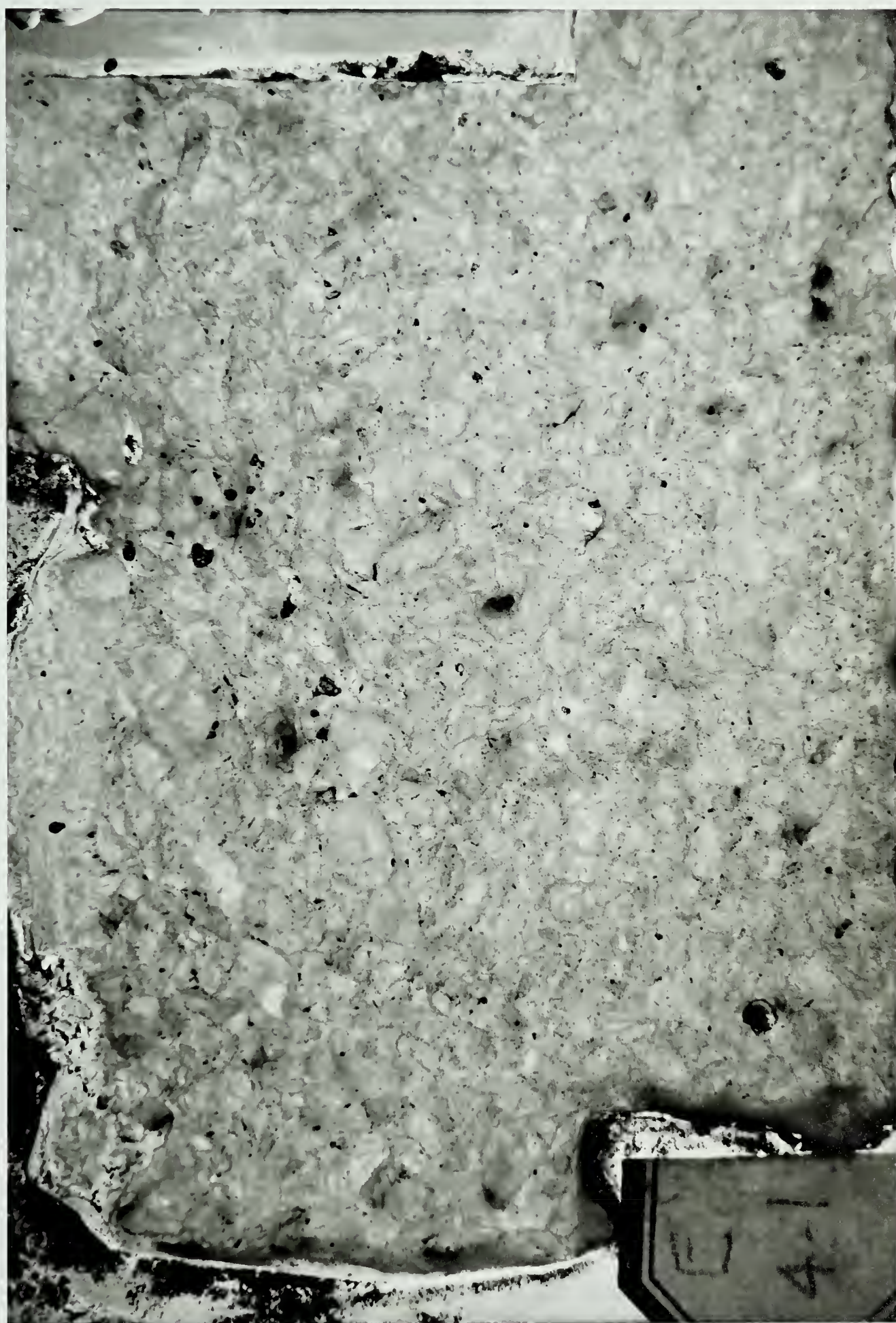


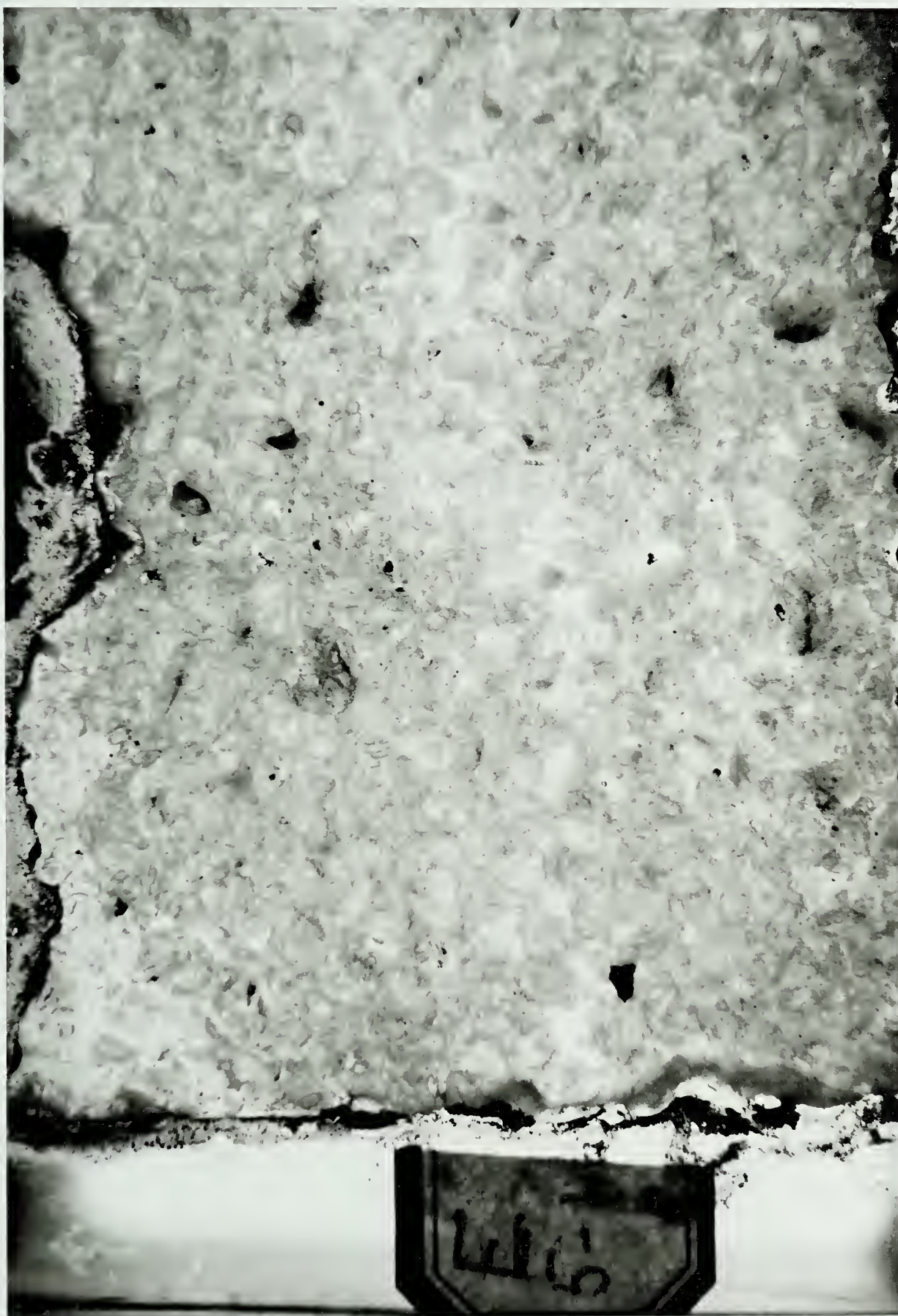












APPENDIX D

STRESS-DISPLACEMENT PLOTS AND SAMPLE DATA SHEETS

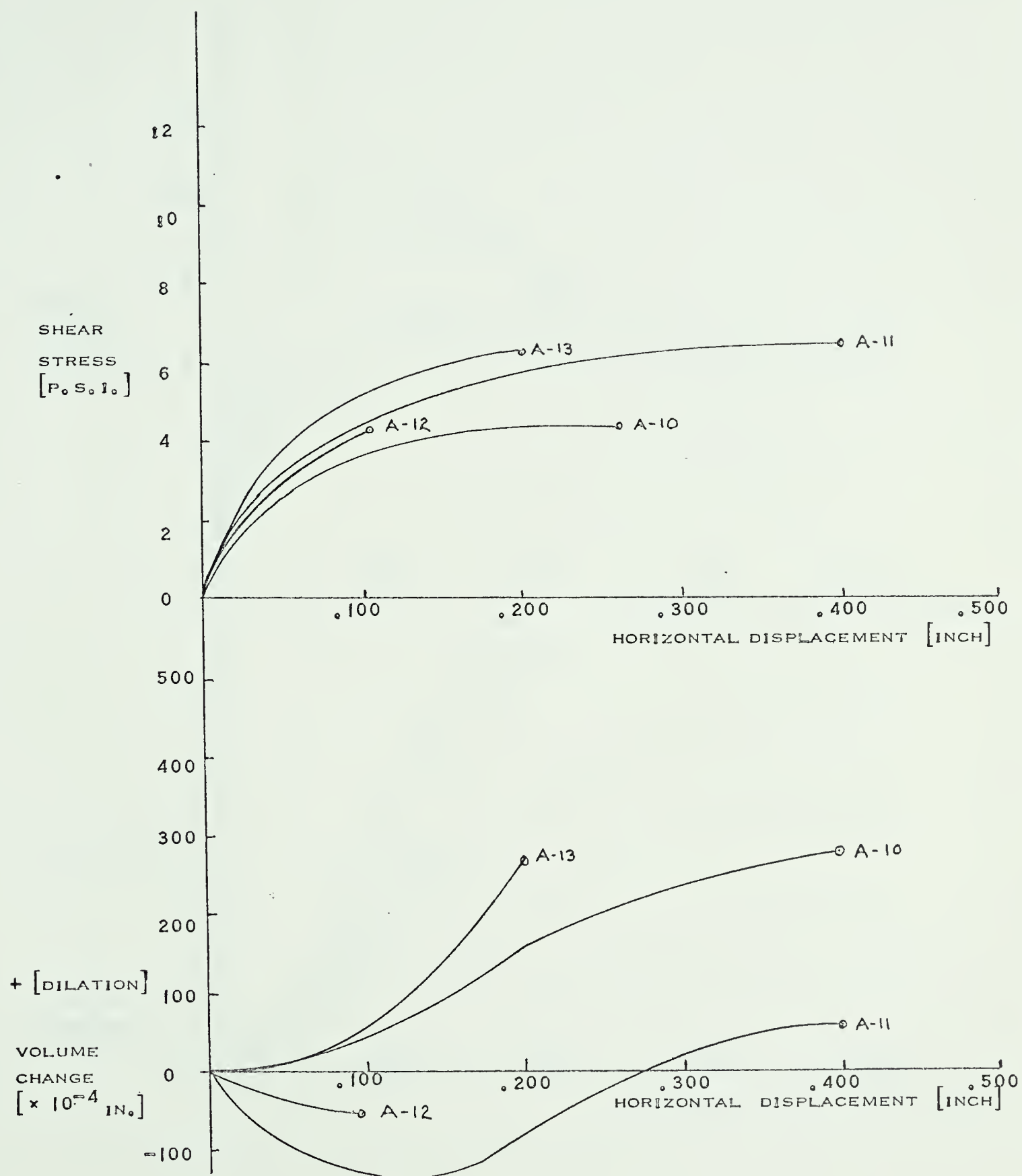


FIGURE D.1 STRESS AND VOLUME CHANGE - DISPLACEMENT CURVES FOR TEST SERIES A

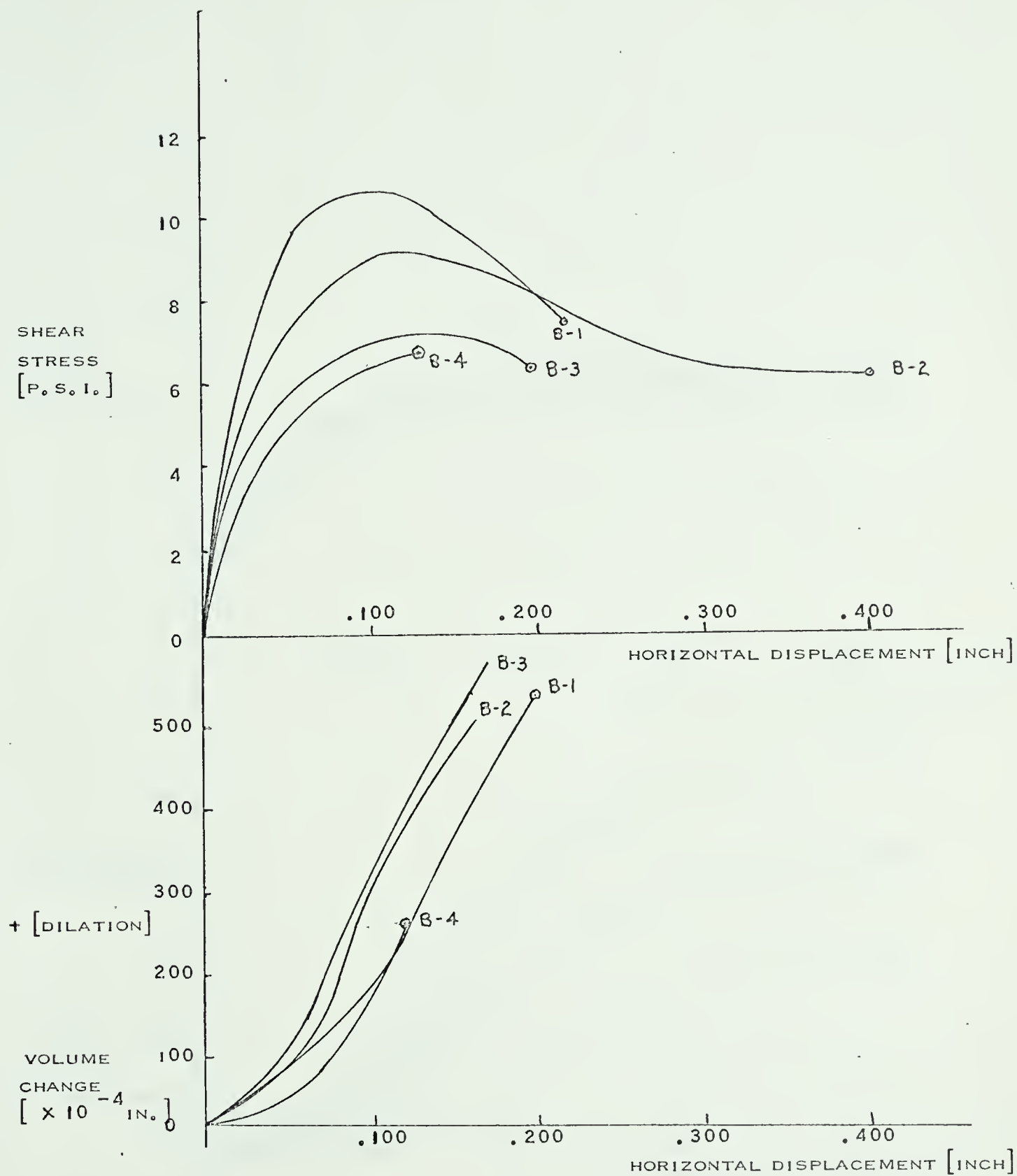


FIGURE D.2 STRESS AND VOLUME CHANGE - DISPLACEMENT CURVES FOR TEST SERIES B

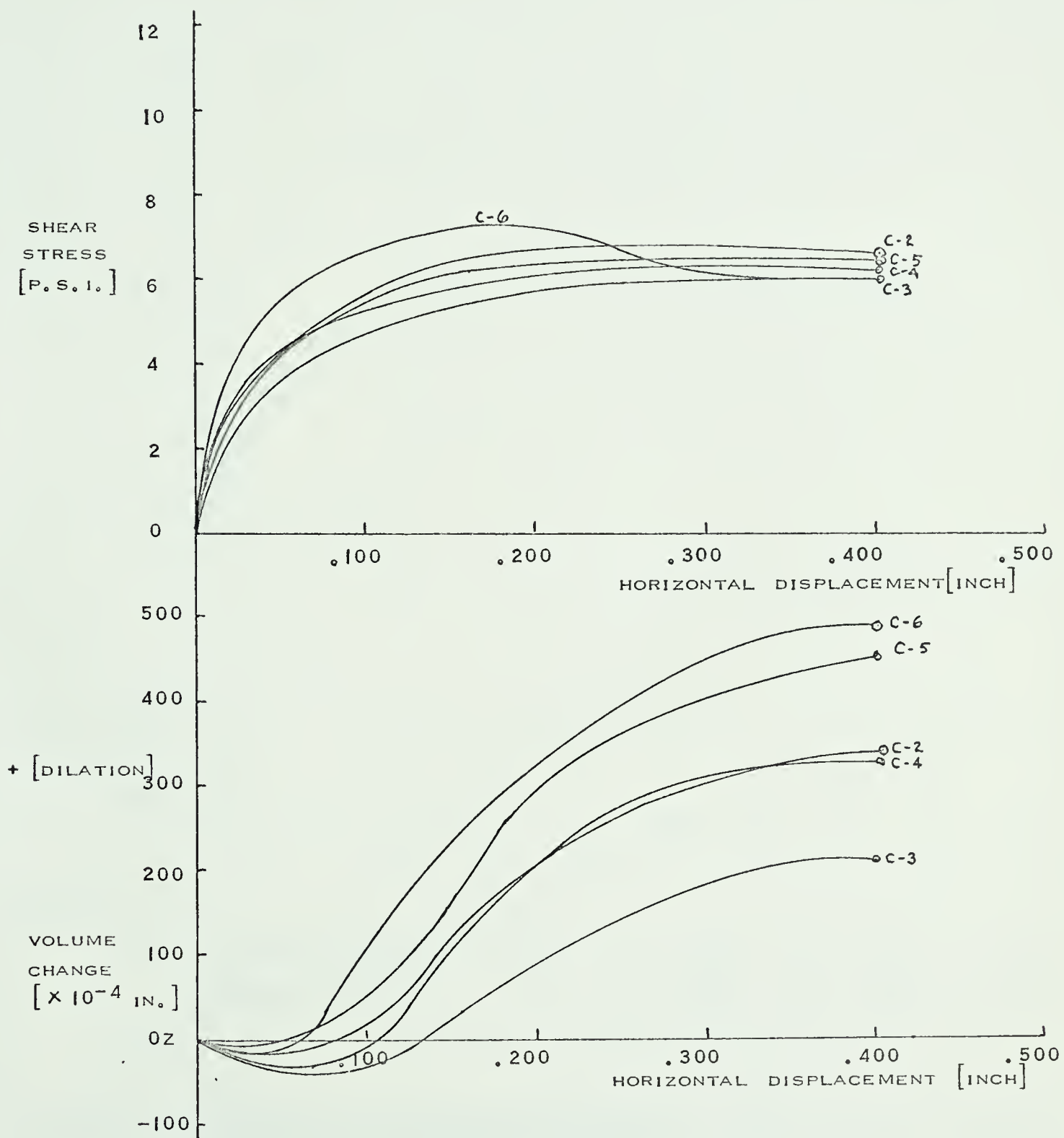


FIGURE D.3 STRESS AND VOLUME CHANGE- DISPLACEMENT CURVES FOR TEST SERIES C

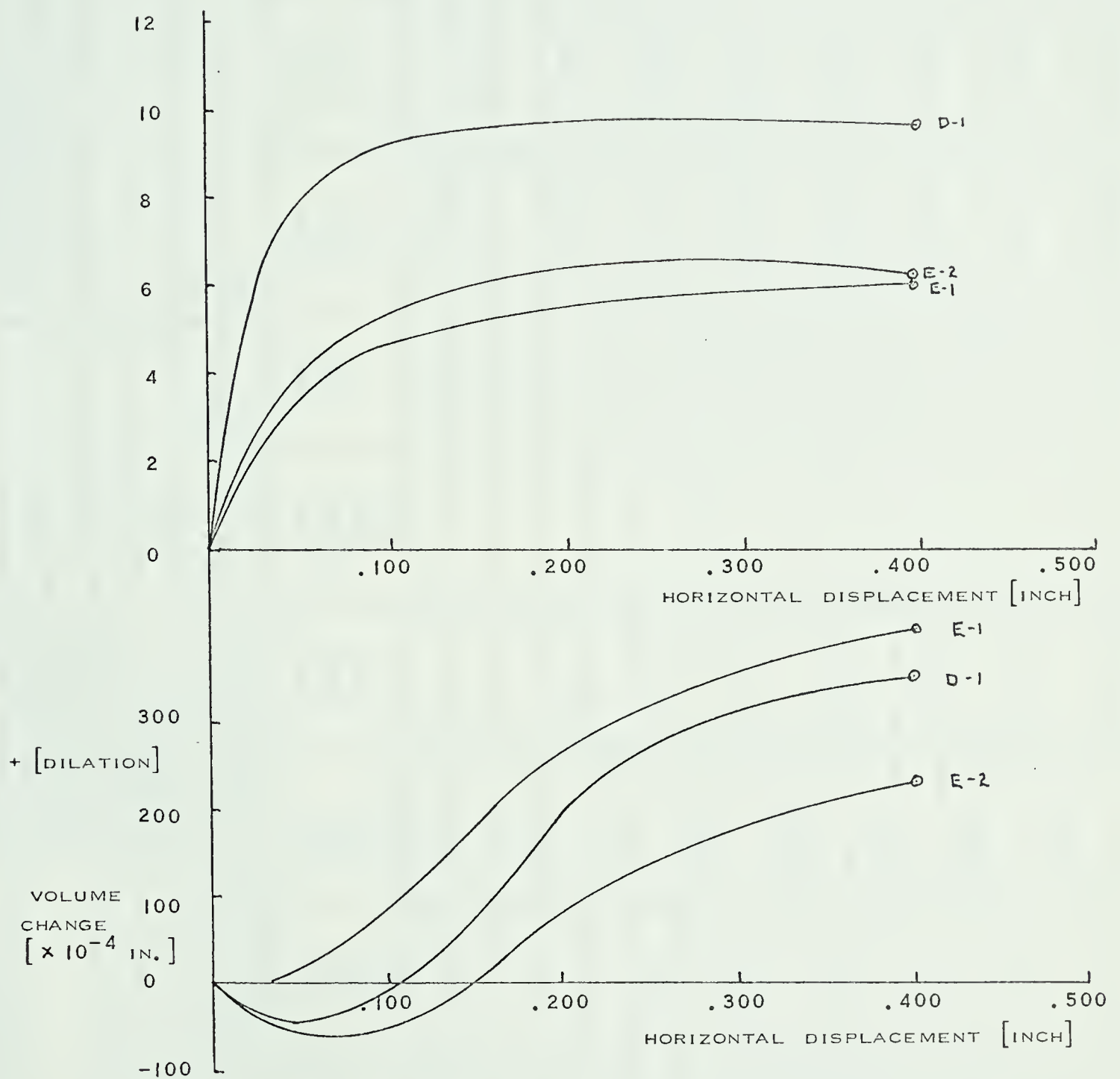


FIGURE D.4 STRESS AND VOLUME CHANGE - DISPLACEMENT CURVES FOR TEST SERIES D AND E

UNIVERSITY OF ALBERTA
DEPARTMENT OF CIVIL ENGINEERING
SOIL MECHANICS LABORATORY
DIRECT SHEAR TEST

PROJECT THESIS
SITE
SAMPLE TRIAL NO. A-10
LOCATION
HOLE DEPTH
TECHNICIAN D.S.M. DATE APR. 10/68

SHEAR BOX:		INITIAL GROSS WEIGHT	354.16 gm.	SAMPLE DESCRIPTION	CRUSHED CALCITE
WIDTH 2.36 in.		FINAL GROSS WEIGHT	165.28 gm.	#10 - #20 SIEVE - LOOSE PACKING	
LENGTH 2.37 in.		WEIGHT OF SPECIMEN	189.58 gm.		
INITIAL AREA 5.59 in. ²		VOID RATIO, e	.92		
HEIGHT OF SPECIMEN 1.89 in. 38 = 1.47 in.		TOTAL NORMAL LOAD	24.8 lb. + 4032 gm.	Gs	2.71 Dio mm
VOLUME OF SPECIMEN 134.7 cm. ³ / 8.22 in. ³		UNIT NORMAL LOAD σ	6.03 p.s.i.	Cu	

Horizontal Base Movement		Proving Ring Dial Reading	Vertical Dial Reading $\times 10^{-4}$ in.	Vertical Strain LOAD (kg.)	Total Horizontal Shear Strain	Sample Length in.	Cross-Section Area in. ²	Total Shear Force lb.	Shear Stress τ p.s.i.	$\frac{\tau}{\sigma}$
Turns	Inches									
0	0	0	0	0		2.37	5.60	0	0	
3	.024	13		4.8		2.35	5.55	10.6	1.9	
6	.048		+08			2.32				
8	.064	21		7.7		2.31	5.45	16.9	3.1	
10	.080		+30			2.29				
12	.096	24		8.9		2.27	5.36	19.6	3.7	
14	.112		+65			2.26				
16	.128	25		9.3		2.24	5.29	20.5	3.9	
18	.144		+100			2.23				
20	.160	27		10.0		2.21	5.22	22.05	4.2	
22	.176		+135			2.19				
24	.192	27		10.0		2.16	5.10	22.05	4.3	
26	.208		+165			2.15				
28	.224	27		10.0		2.14	5.07	22.05	4.3	
30	.240		+205			2.13				
32	.256	27		10.0		2.11	4.98	22.05	4.4	
34	.272		+225			2.10				

UNIVERSITY OF ALBERTA
DEPARTMENT OF CIVIL ENGINEERING
SOIL MECHANICS LABORATORY

DIRECT SHEAR TEST

PROJECT THESIS
SITE
SAMPLE TRIAL NO. B-1
LOCATION
HOLE DEPTH
TECHNICIAN D.S.M. DATE APR 23/68

SHEAR BOX:		INITIAL GROSS WEIGHT		SAMPLE DESCRIPTION	
WIDTH	<u>2.36 IN.</u>	INITIAL GROSS WEIGHT		CRUSHED CALCITE	
LENGTH	<u>2.37 IN.</u>	FINAL GROSS WEIGHT		#10 - #20 SIEVE SIZE, DENSE	
INITIAL AREA	<u>5.59 IN.</u>	WEIGHT OF SPECIMEN	<u>189.6 GM.</u>	PACKING	
HEIGHT OF SPECIMEN	<u>1.89 - .62 = 1.27 IN.</u>	VOID RATIO, e	<u>.65</u>		
VOLUME OF SPECIMEN	<u>116.3 CM.³</u>	TOTAL NORMAL LOAD	<u>24.8 LB. + 4032 GM.</u>	Gs <u>2.71</u> Dio mm	
		UNIT NORMAL LOAD	σ <u>6.03 P.S.I.</u>	Cu	

Horizontal Base Movement		Proving Ring Dial Reading	Vertical Dial Reading $\times 10^{-4}$ IN.	Vertical Strain INCH	Total Horizontal Shear Strain	Sample Length IN.	Cross-Section Area IN. ²	Total Shear Force KG.	Shear Stress τ P.S.I.	$\frac{\tau}{\sigma}$
Turns	Inches									
0	0	0	0			2.37	5.60	0	0	
2	.016	32				2.35	5.57	11.9	4.7	
4	.032		+ 58			2.34	5.55			
6	.048	62				2.32	5.51	23.0	9.2	
8	.064		95			2.31	5.47			
10	.080	70				2.29	5.43	25.9	10.5	
12	.096		162			2.27	5.40			
14	.112	72				2.26	5.36	26.7	11.0	
15	.120		200			2.25	5.33			
16	.128	70				2.24	5.30	25.9	10.8	
17	.136		288			2.23	5.28			
18	.144	65				2.23	5.25	24.1	10.1	
20	.160		400			2.21	5.23			
21	.168		434			2.20	5.21			
23	.184	53	490			2.19	5.19	19.6	8.3	
25	.200		533			2.17	5.14			
27	.216	48				2.15	5.10	17.8	7.7	

DIRECT SHEAR TEST

PROJECT THESIS

SITE

SAMPLE

LOCATION

HOLE

TECHNICIAN D.S.M. DATE 28 MAY/68

PROJECT THESIS

SITE

SAMPLE

LOCATION

HOLE

TECHNICIAN D.S.M. DATE 28 MAY/68

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